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HIGH TEMPERATURE EFFECTS ON BITUMINOUS MIXES

By William H. Gotolski,[1] M. ASCE

SYNOPSIS

Temperature effects on the stability characteristics of bituminous mixtures were studied for four asphalt and aggregate temperatures. This study was made to ascertain the temperature at which detrimental characteristics are exhibited. Analysis of variance techniques were used to evaluate the basic factors affecting the test results.

INTRODUCTION

Two of the essential properties required of bituminous paving mixtures are stability and durability. The temperatures to which an asphalt-aggregate combination may be exposed during its processing, without detrimental effects on stability and durability, remains a controversial matter among designers of bituminous pavements. This study was undertaken to investigate the effects of high temperatures, which may result from overheating of either the asphalt or the aggregate or both, on the structural properties of the bituminous mixes and the rheological properties of the asphalt cement used in the mixes. The structural or stability characteristics of the bituminous mixtures were measured by the Marshall method and the Hveem Stabilometer.

Temperatures used in the study were 275°, 350°, 425°, and 500° F. The rheological or physical characteristics of asphalt cements found to be of importance for durability and long "life," penetration, softening point, and ductility, were

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measured prior to test temperatures and after being subjected to test temperatures. It was the intent of this investigation to establish the highest temperature to which an asphalt-aggregate combination could be exposed without critically decreasing its stability characteristics. Further, an attempt was made to postulate the end effect, that is, to determine whether the temperature chosen would seriously shorten the "life" of the pavement or result in greater loss of durability.

The physical properties, penetration, softening point, and ductility were considered in this study, as earlier investigations[2,3,4,5] found significant relationships existed with pavement durability.

Materials used in this study were crushed limestone aggregate and limestone dust and an 85/100 penetration asphalt cement. The investigation was limited to a laboratory controlled experiment and used a statistical method known as analysis of variance, to evaluate the collected data.

EXPERIMENTAL DESIGN

Studies concerning the durability of portland cement concrete or soil-cement mixtures, or the permeability of different concretes, or the compressive strengths of specimens from different concrete-mix designs, or the stability of asphaltic concrete mixes, are particularly suited to the statistical procedure known as the analysis of variance. Such an analysis eliminates much personal bias in interpreting the results. Inferences made from data are based on mathematical probabilities.

The experimental design may be considered as the blueprint for the manner in which data may be collected on one or more variables. The experimental design used in this study was patterned after such designs appearing in available references on experimental design and statistical analysis.[6,7]

The two independent variables in this study, asphalt temperature and aggregate temperature, were chosen because of their importance in practice. Each increment of these variables was purposely made coarse (75° F) so that any significance or interaction discovered could be considered as acting over a broad range of a variable. In order to obtain the information desired, an experimental design for a two-factor classification of data was adopted. In Table 1 the X's in the blocks may be stability measures or flow values, depending on which is to be analyzed. In the study of the effect of high temperatures on the stability of asphalt mixes, the variables of classification were asphalt temperatures (A) and aggregate temperatures (S). Here A_1, A_2, A_3, and A_4 represent asphalt masses (5 gallons) previously heated to 275°, 350°,

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2 "Some Low-Temperature Characteristics of Bituminous Paving Compositions," by H. W. Skidmore, Transactions, ASCE, Vol. 101, 1936, p. 1135.

3 "Asphalt Volatility and Weathering Tests," by R. G. Clark, Proceedings, The Association of Asphalt Paving Technologists, Vol. 25, 1956, pp. 417-431.

4 "A Study of Asphalt Recovery Tests and Their Value as a Criterion of Service Behavior," by V. B. Steinbaugh and J. D. Brown, Proceedings, The Assn. of Asphalt Paving Technologists, Vol. 9, 1937, p. 138.

5 "Measurement of the Resistance of Oil Asphalts (50/60 pen) to Changes in Penetration and Ductility at Plant Mixing Temperatures," by C. L. Shattuck, Proceedings, The Assn. of Asphalt Paving Technologists, Vol. 11, 1940, p. 186.

6 "Applied Statistics for Engineers," by W. Volk, McGraw-Hill Book Company, Inc., New York, 1958.

7 "Application of Statistical Methods to Laboratory Concrete Freeze-Thaw Test Data," by P. E. Irick and J. B. Blackburn, Proceedings, Highway Research Bd., Vol. 34, 1955, p. 329.

425°, and 500° F, respectively. The low temperature was chosen to satisfy the lower limit set by mix specifications. The upper limit of 500° F, somewhat below the flash point of the material, was chosen for safety reasons. Each asphalt sample was maintained at its particular temperature for a period of 30 min. They were then coded as A_1, A_2, A_3, and A_4, and stored in closed cans until they were to be used in a mix.

This first heating of the asphalt was carried out to study the effect of temperature on the rheology of the asphalt cement when it was heated in relatively large masses.

In the experimental design S_1, S_2, S_3, and S_4 represent aggregate heated to 275°, 350°, 425°, and 500° F, respectively, prior to being combined with the required amount of reheated asphalt cement.

The data collected on the stability of the specimens was then analyzed by using the "F" ratio.[6] The "F" ratio is the statistic in the analysis of variance

TABLE 1.—TWO CLASSES OF DATA WITH REPLICATION

	A_1	A_2	A_3	A_4
S_1	X_{1111} X_{1112} X_{1113}	X_{2111} X_{2112} X_{2113}	X_{3111} X_{3112} X_{3113}	X_{4111} X_{4112} X_{4113}
S_2	X_{1211} X_{1212} X_{1213}	X_{2211} X_{2212} X_{2213}	X_{3211} X_{3212} X_{3213}	X_{4211} X_{4212} X_{4213}
S_3	X_{1311} X_{1312} X_{1313}	X_{2311} X_{2312} X_{2313}	X_{3311} X_{3312} X_{3313}	X_{4311} X_{4312} X_{4313}
S_4	X_{1411} X_{1412} X_{1413}	X_{2411} X_{2412} X_{2413}	X_{3411} X_{3412} X_{3413}	X_{4411} X_{4412} X_{4413}

used to estimate significance of the main effects or interaction. Critical values of "F" are obtained from tables of the "F" distribution. In this study, the five-percent level was used, which is to say that in not more than five cases in one hundred would such an "F" ratio occur if the variances were equal or from the same normal population.

Further information regarding the analysis of variance technique may be found in Volk[6] and Irick and Blackburn.[7]

MATERIAL USED

All of the specimens tested were prepared using standard aggregates and asphalt cement. To eliminate the variables introduced by different sources of

aggregate and asphalt, crushed limestone and limestone dust from a single local source was blended with an 85/100 penetration-grade asphalt cement, also obtained from a single source.

The aggregate used was clean, sound, tough hard stone, free of dirt or foreign matter. The Los Angeles abrasion test indicated that the percent of wear was less than 35 after 500 revolutions. Other pertinent information concerning physical characteristics of the aggregate are: toughness, greater than 6; sodium sulfate soundness, less than 10% loss by weight at 5 cycles; and there was less than 5% by weight of thin and elongated pieces of aggregate. These test results indicated that the aggregate satisfied the state of Pennsylvania's Department of Highways specifications for hardness, particle shape, and sodium sulfate soundness. Gradation checks of the quarry stockpile indicated the gradation to fall within the department's specifications. Specific gravities of the aggregate were determined by the American Society for Testing Materials (ASTM) approved methods. The average specific gravities of the materials were: coarse aggregate, 2.712; fine aggregate, 2.682; and mineral filler, 2.709.

LABORATORY EXPERIMENTAL WORK

The laboratory work consisted of the preparation of 192 specimens made with crushed limestone and limestone dust. Representative samples of the aggregates were taken from quarry stockpiles, oven dried, and separated into nine sizes using the 1/2 in., 3/8 in., Nos. 4, 8, 16, 30, 50, 100, and 200 mesh sieves.

Two different bituminous mixtures were used to study the effects of temperature on the stability of the specimens. The gradations are:[8]

1. The Pennsylvania State Highway Department Specifications, ID-2, surface course, maximum size aggregate—1/2 in.
2. The Pennsylvania State Highway Department Specifications, FJ-1, surface course, maximum size aggregate—3/8 in.

For the purposes of this report, these mixes are designated as Mixes C and F, respectively. The gradations used in this study are given in Table 2.

The asphalt cement used in the preparation of the specimens was obtained from a single source in one shipment and confronted to the Pennsylvania Department of Highways specifications. The mixes were molded at an asphalt content of 6% by weight of the total weight of the mixtures for the gradations selected. Properties of the original asphalt sample and of asphalt masses heated to the four test temperatures are shown in Table 3.

The quantity of combined aggregate to make one specimen having dimensions approximately 2.5 in. high and 4 in. in diameter was determined and the various sizes carefully recombined to make this weight. The dry aggregate was stirred to produce a uniform blend and then placed in a forced draft oven.

After the proper gradation was obtained, the total blend was heated to prescribed test temperatures of either 275°, 350°, 425°, or 500° F. Just prior to combining the aggregate and the proper amount of a coded asphalt, the asphalt was heated[8] to a temperature of 275 $\pm$ 5° F.

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8 "Specifications for Plant Mix," Dept. of Highways, Commonwealth of Pa., Bulletin No. 27, Rev. 1954, Harrisburg, Pa.

The heating of equipment followed standard procedures as outlined[9] by the Asphalt Institute for the Marshall method. The Marshall method of compaction was used because of availability of such laboratory facilities and equipment. Each specimen was compacted by applying 50 blows to each face with the Marshall hammer. This amount of compaction was applied as this is the compaction normally used for mixes designed for traffic using tire inflation pressures of 100 psi and in the design[9] of highway pavements.

Upon completion of compaction, the specimens were then allowed to cool to room temperature. The specimens were then labeled and their dimensions measured. Specific gravity determinations were made by weighing the test specimens in air and in water.[10] The specimens were then stored on a smooth, level surface until ready for further testing.

Asphaltic concrete mixtures are commonly tested at a temperature of 140° F. Practice varies with regard to the manner in which the 140° F temperature is produced. One of the methods provides for obtaining the temperature by heating in a water bath, while another requires the specimens to be heated in an

TABLE 2.—GRADATIONS OF AGGREGATES USED IN ASPHALTIC CONCRETE MIXES

Passing Sieve (1)	Sieve Size, mm. (2)	Gradation Mix C		Gradation Mix F	
		Spec. % (3)	Design % (4)	Spec. % (5)	Design % (6)
1/2-in.	13.33	100	100	-	-
3/8-in.	9.52	80-100	90	100	100
#4	4.72	45-72	58.5	90-100	95
#8	2.38	30-55	42.5	70-100	85
#16	1.19	20-40	30	40-80	60
#30	0.59	10-30	20	20-60	40
#50	0.297	5-20	12.5	5-40	22.5
#100	0.149	2-12	7	0-20	10
#200	0.074	2-8	5	0-10	5

oven. All of the specimens in this investigation, prior to the measurement of their stability, were heated for a period of at least one hour in a constant temperature water bath held at 140 ± 1° F. The specimens were in plastic bags while in the water bath. In all cases, the specimens were tested as quickly as possible after removal from the bath in order to minimize loss of heat.

Upon removal from the water bath, the specimens were carefully dried and tested in the Marshall apparatus. Further details of the Marshall test were in accordance with standard procedure.[9]

Test specimens for the Hveem stability test were prepared in the same manner as the Marshall test specimens. After the test specimens had remained in a 140° F water bath for a minimum of 1 hr, further testing in the Hveem Stabilometer was in accordance with standard procedure[9] with three exceptions as follows:

1. Before a test specimen was removed from the water bath, a strip of adding machine paper (2-3/4 in. wide, 13-1/4 in. long) was slit transversely every 3/4 in. to within 1/4 in. of the edge, and rubber cement was applied to one

9 "Mix Design Methods for Hot-Mix Asphalt Paving," The Asphalt Inst., Manual Series No. 2, 1st Ed., College Park, Md., April, 1956.

10 "Statistical Quality Control Applied to an Asphalt Mixing Plant," by F. B. Odasz, Jr. and D. R. Nafus, Proceedings, The Assn. of Asphalt Paving Technologists, Vol. 23, 1954, p. 78.

end of the strip. The test specimen was then removed from the water bath and this strip of paper firmly wrapped around the dry specimen. This procedure was necessary to protect the rubber diaphragm of the stabilometer from damage due to the sharp aggregate particles in the specimens.

2. 4 in. paper discs were placed on the top and bottom of the test specimen before the specimen was lowered into the stabilometer chamber. The stabilometer stage was adjusted if necessary to conform to the height of the specimen.

3. The load was applied at a rate of 0.05 in. per min, and readings of lateral pressure and vertical deformation were made at vertical loads of 500 lb, 1,000 lb, 1,500 lb, 2,000 lb, 2,500 lb, and 3,000 lb and at each 1,000 lb of vertical load thereafter. The vertical load most nearly approaching a lateral pressure of 200 psi was also recorded.

The vertical load was then reduced to 1,000 lb and the displacement necessary to raise the lateral pressure from 5 psi to 100 psi was obtained (the final

TABLE 3.—PROPERTIES OF ASPHALT CEMENT

	Sample Identification				
	Original	A_1	A_2	A_3	A_4
Specific Gravity at 77°F.	1.021	1.021	1.021	1.021	1.021
Softening Point (Ring and Ball) °F.	117	113	113.5	113.5	115
Penetrations: 200 gms., 60 secs., at 32°F.	22				
100 gms., 5 secs., at 77°F.	96	94	92	91	89
50 gms., 5 secs., at 115°F.	350+				
Ductility, 5 cm./min. at 77°F., cms.	150+	150+	150+	150+	150+
Flash Point, C.O.C., °F.	575				
Loss on heating: 50 gms., 5 hrs. at 325°F., wt. %	0.06				
Per cent of original penetration	81.3				
Solubility, in CCl_4, wt. %	99.95				
in 86° Naptha, wt. %	80.5				
Sulfur Bomb method, wt. %	3.20				
Viscosity, Saybolt Furol at 275°F., secs.	190				
at 350°F., secs.	35				

displacement). Table 4 shows a typical set of data obtained from the test of a specimen.

EXPERIMENTAL RESULTS

To evaluate the effects of high temperatures on the stability of bituminous mixes, the Marshall method and the Hveem stabilometer were used. The stability values as measured were corrected by using charts that were readily available.[9] The final test data, percent voids, flow, stability, and so on, are presented in Tables 5 and 6, of the Appendix.

The analysis of variance technique was then used to see whether the asphalt and aggregate temperature treatments together, or either alone, would significantly affect the stability of the molded specimens. The numerical results of the analysis of variance calculations for the Marshall stability of Mix C are presented in Tables 7, 8, and 9, of the Appendix.

Statistically it is seen that the two main effects, asphalt temperature and aggregate temperature, were significant with regard to the stability measure. This is to say that in the overall procedure of preparing and testing the molded specimens, the effects of the variables, asphalt temperature and aggregate temperature, were causes for significant variations in the stability measure. The interaction term was not significant in this analysis. This means that the two main effects were operating independently of each other. The two main effects and the interaction were not significant with respect to the flow value.

Cause-effect results may be physically developed by citing the physical characteristics measured by the tests used in this study. The Marshall stability value is said[11] to be primarily a measure of cohesion and viscous resistance of the bituminous binder. The relationship of stability and asphalt temperature indicates that a mass of asphalt exposed to increasingly higher

TABLE 4.—HVEEM STABILOMETER TEST

Vertical Load, in pounds (1)	Hveem Gauge, in psi (2)	Vertical Strain, in inches (3)	Mix C Specimen No. 2314 (4)
0	5	--	Height of Specimen - 2.5937"
100	7	0.00	
500	11	0.22	Displacement turns:
1,000	15	0.31	Initial 2.04
1,500	18	0.37	Final 2.24
2,000	21	0.41	
2,500	25	0.45	$S = \frac{22.2}{\frac{P_h D_2}{P_v - P_h} + 0.222}$
3,000	29	0.49	
4,000	38	0.55	
5,000	49	0.60	
6,000	63	0.66	
7,000	80	0.71	$S = \frac{22.2}{\frac{49 \times 2.24}{400 - 49} + 0.222}$
8,000	101	0.77	
9,000	124	0.82	
10,000	149	0.87	
11,000	177	0.93	S = 41.5

temperatures results in some change in the cohesion and viscosity of the binder. Further computations show this to be a detrimental effect on both Mix C and Mix F.

In the relationship of stability and aggregate temperature, it is indicated that increasing aggregate temperature results in changes in stability. Since the aggregate usually constitutes about 95% of the total mix, the mix temperature is largely determined by the aggregate temperature. At higher temperatures of aggregate, there would be a better coating of the aggregate by the binder and a more uniform distribution of asphalt. This, coupled with better compaction at higher temperatures, would result in changes of stability with aggregate temperature. The Marshall stability also measures to some extent the mechanical stability of the aggregate.

The flow index is said[11] to be an indirect measure of internal friction of the aggregate. From analyses made to check flow against either asphalt or aggregate temperatures, it was found that flow remained relatively unchanged. Each

11 "Bituminous Paving Mixtures," Bulletin No. 105, Highway Reseach Bd., 1955.

specimen had the same aggregate and gradation, so that the internal friction should remain relatively the same at all test temperatures.

The statistical analysis used in analyzing the data of Mix F also indicated significance of aggregate temperature on stability and on flow measure. This implies that the aggregate temperature was the cause of variation in the stability and flow of the final mixture. Neither asphalt temperature nor the interaction of the two main effects was significant. The physical cause of stability variation with aggregate temperature has been previously mentioned. The physical effect of variation of flow may be caused not only by temperature changes but also by the gradation of the mix itself. Mix F contains a larger percentage of the smaller sized particles which may cause variation in coating of aggregate by the binder. This in turn would possibly cause some variation in the internal friction of the mix.

In the analysis of the data obtained from the Hveem stabilometer test, it was found that only the main effect of asphalt temperature treatment was significant for both mixes. This indicates that, statistically speaking, the asphalt temperature treatment was a cause of significant variation in the stability measure. Because the interaction and other main effects were not significant, the asphalt temperature was an independent operator.

Physically there appears to be no cause-effect relation in that the Hveem relative stability is said[11] to depend almost entirely on internal friction of the aggregate and is influenced to only a slight degree by the cohesion or viscous resistance of the bituminous binder in the mixture. From these analyses, it would appear that all of the changes produced were not revealed or detected by any one measure of stability. This is as might be anticipated. The two stability tests employed are empirical and are different in their modes of measuring stability. The variation of the Hveem stability as measured at the test temperatures are relatively small, and, as indicated by the relations to be established, may have little practical significance.

In analyzing the data obtained by the Marshall method, the stability and flow measures were related to both asphalt and aggregate temperatures. When the stability measures obtained on Mix C were related to asphalt temperature, a linear relationship was seen to exist. Further analysis showed that this relationship can be expressed by one general equation. The F-tests indicated no further variation in quadratic or cubic trends.

The correlation computations made[6] indicate that this relationship may be expressed as

$$y = 1858.92 - 0.46\,x \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

This equation is plotted in Fig. 1(a), together with the original data.

The analysis of variance applied to stability and its relations to aggregate temperatures indicated the quadratic component to be significant. Analysis of the specific components of the interaction revealed no significance. Thus, a single quadratic equation can represent the experimental data. The quadratic relationship may be expressed as

$$y = 560.79 + 4.27\,x - 0.003385\,x^2 \quad \ldots\ldots\ldots\ldots \quad (2)$$

and is plotted in Fig. 1(b), together with the original data.

The flow values of Mix C were analyzed in the same fashion. The slopes of the lines relating flow to asphalt or aggregate temperature were not significantly different from zero. Thus, the relations can be said to remain constant

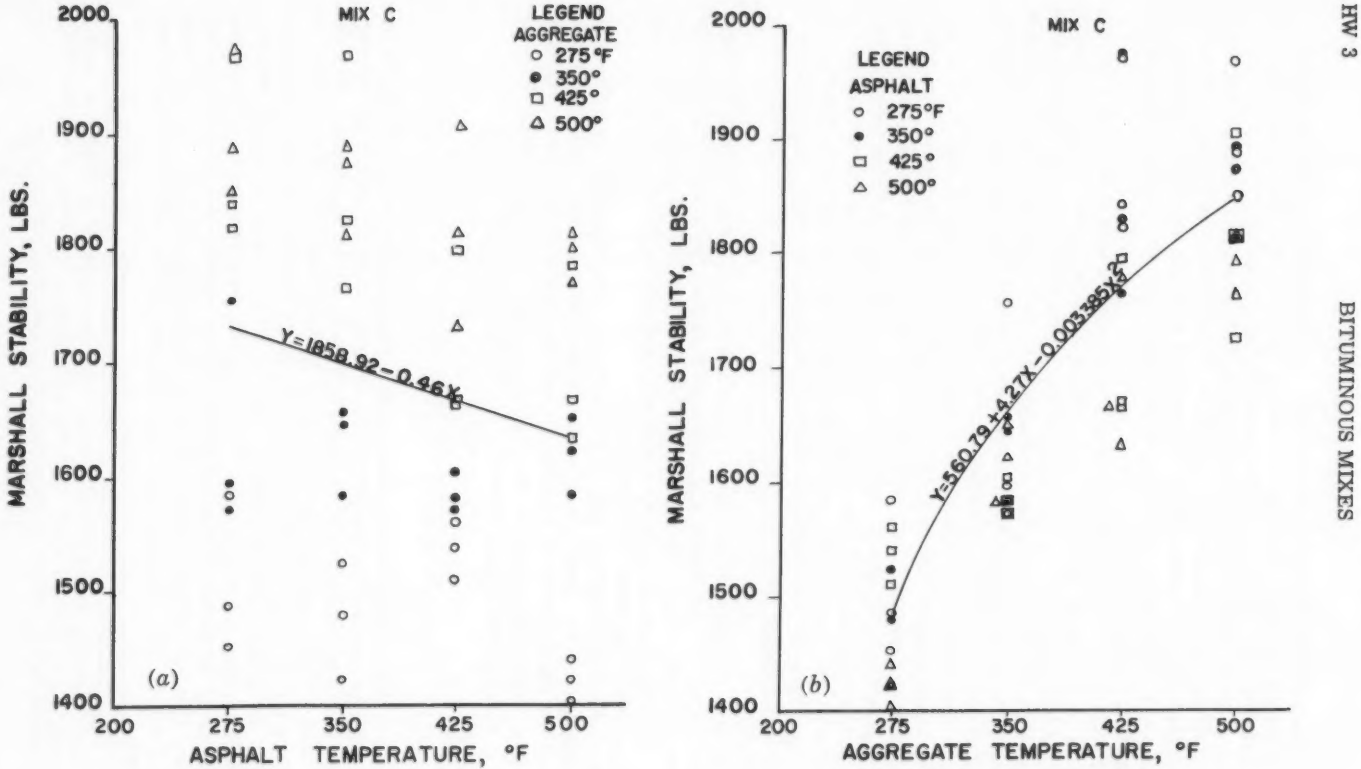

FIG. 1.—MARSHALL STABILITY—TEMPERATURE RELATIONS

for the temperature ranges investigated. The experimental results appear in Fig. 2. Using similar analyses on Mix F data, the results are as shown in Figs. 3 and 4. For all figures the legend of Fig. 1 applies.

From a study made of these relationships, it is seen that a general decrease in stability will result in mixes in which the asphalt has been heated to increasingly higher temperatures. This is generally true for both Mix C and Mix F. On the other hand, it is seen that higher stability is the result of heating the aggregate to higher temperatures.

The data compiled through the Hveem stabilometer test procedure was also analyzed in the same manner. The results are presented in Figs. 5 and 6. Fig. 6(a) illustrates the data collected in the study concerned with relative stability and the temperature to which asphalt had been exposed in the overheating process. Having no explanation for such data, the writer suggests that they be viewed as data until further studies can be made.

Mix F indicated no significant differences in the study of stability and aggregate temperature. The analysis also indicated that the slope of the relationship was not significantly different from zero. Thus, the results are the same as was found for Mix C. The experimental results indicate that the stability remains constant over the ranges of aggregate temperatures studied. In other words, the stability for both gradations is not significantly affected, by the temperature of the aggregate used in preparing the mix. Apparently, the Hveem stabilometer does not directly indicate the effects of aggregate temperature.

A study of stability measures in relation to asphalt temperature shows that Marshall stability decreases with increasing asphalt temperature; the Hveem relative stability shows increasing values with increasing asphalt temperatures. The apparent contradiction is due to the fact that the two stability tests do not measure the same characteristics.

Other studies[11] indicate this to be the case. The Marshall stability value is a measure of the cohesive or viscous properties of the asphaltic binder, whereas the stabilometer is almost entirely dependent on the internal friction of the aggregate. The effect of the asphalt and aggregate temperatures on the stability measures has been illustrated by Figs. 1 through 6.

ANALYSIS OF TEST RESULTS

While a direct comparison of data with other studies cannot be made, the rheological properties of the preheated asphalt follow the same pattern outlined by Hubbard and Gollomb[12] in their work with 50/60 pen asphalt. The asphalt used in this investigation was heated in mass batches and maintained at their respective temperatures for a period of not less than 30 min. The properties of the asphalt are reported in Table 3. There is no excessive hardening of the asphalt. Comparison of the change in penetration with the original penetration of 96 shows increasing percentage of change with increasing temperature. The percentage changes are 2.1%, 4.2%, 5.2%, and 7.3%, respectively. Comparison of the softening point and ductility values shows no significant changes.

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12 "The Hardening of Asphalt with Relation to Development of Cracks in Asphalt Pavements," by P. Hubbard and H. Gollomb, Proceedings, The Assn. of Asphalt Paving Technologists, Vol. 9, 1937, p. 165.

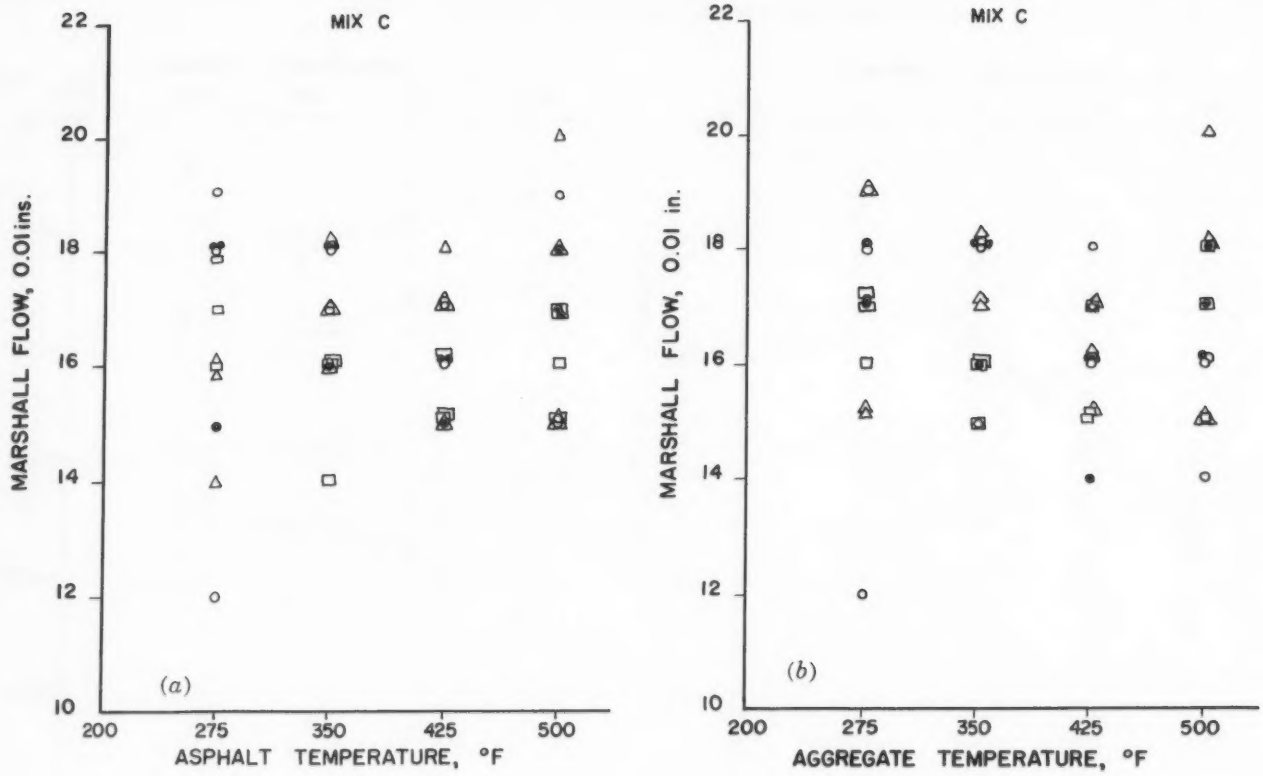

FIG. 2.—MARSHALL FLOW—TEMPERATURE RELATIONS

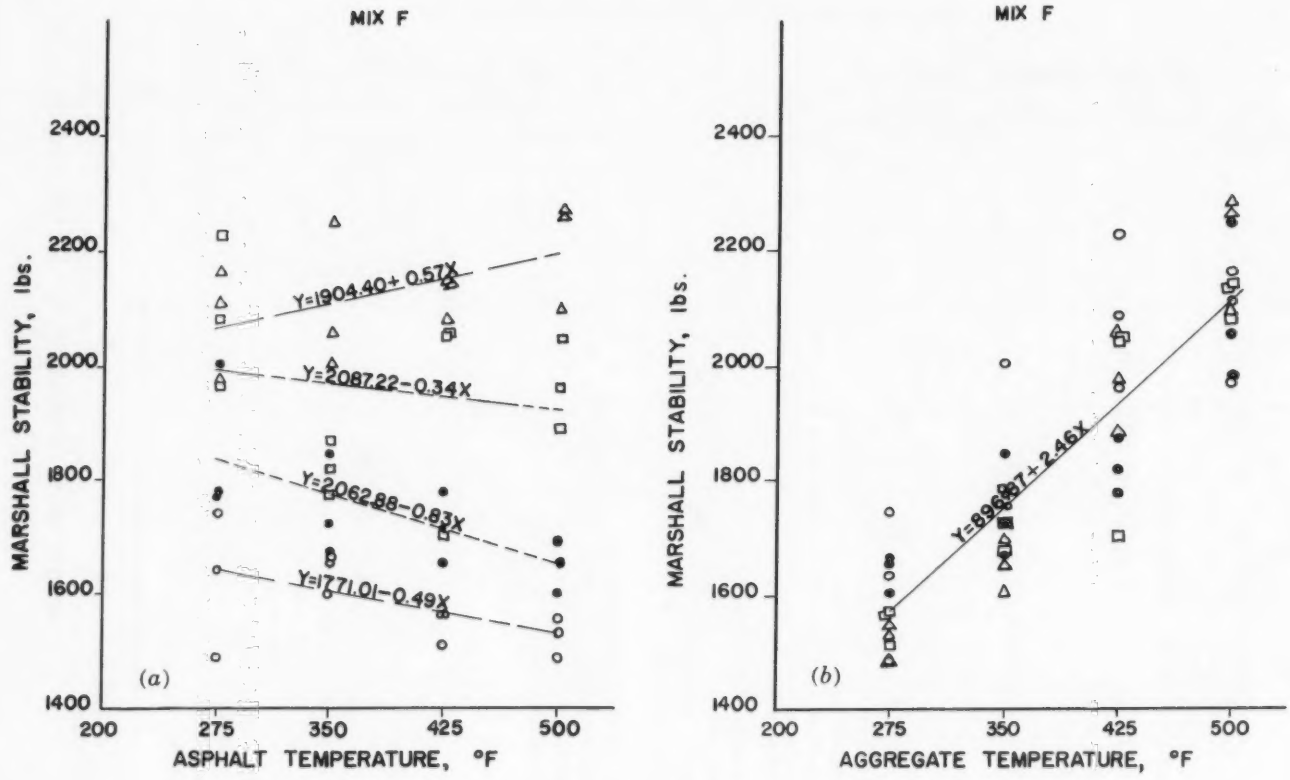

FIG. 3.—MARSHALL STABILITY—TEMPERATURE RELATIONS

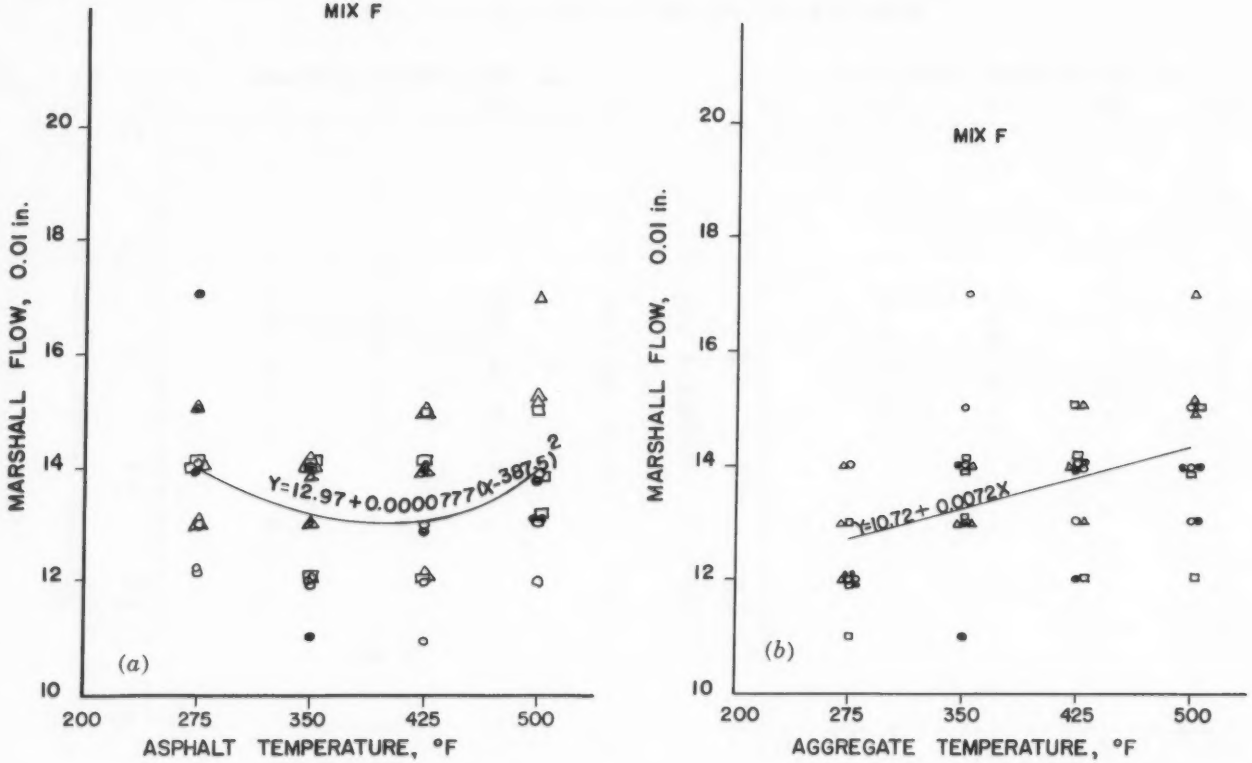

FIG. 4.—MARSHALL FLOW-TEMPERATURE RELATIONS

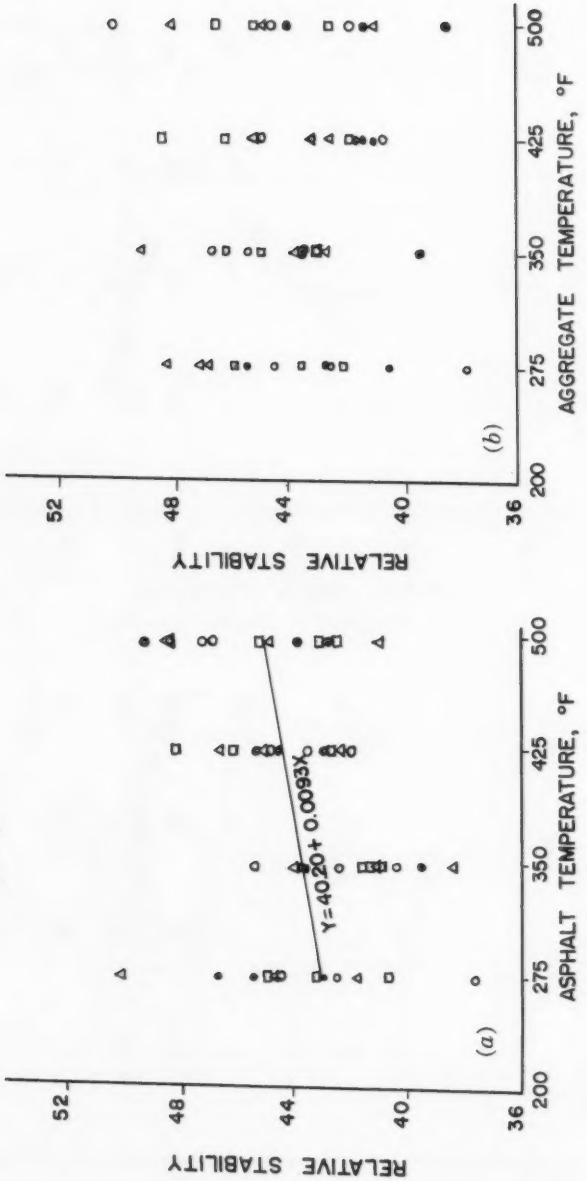

FIG. 5.—HVEEM STABILITY-TEMPERATURE RELATIONS

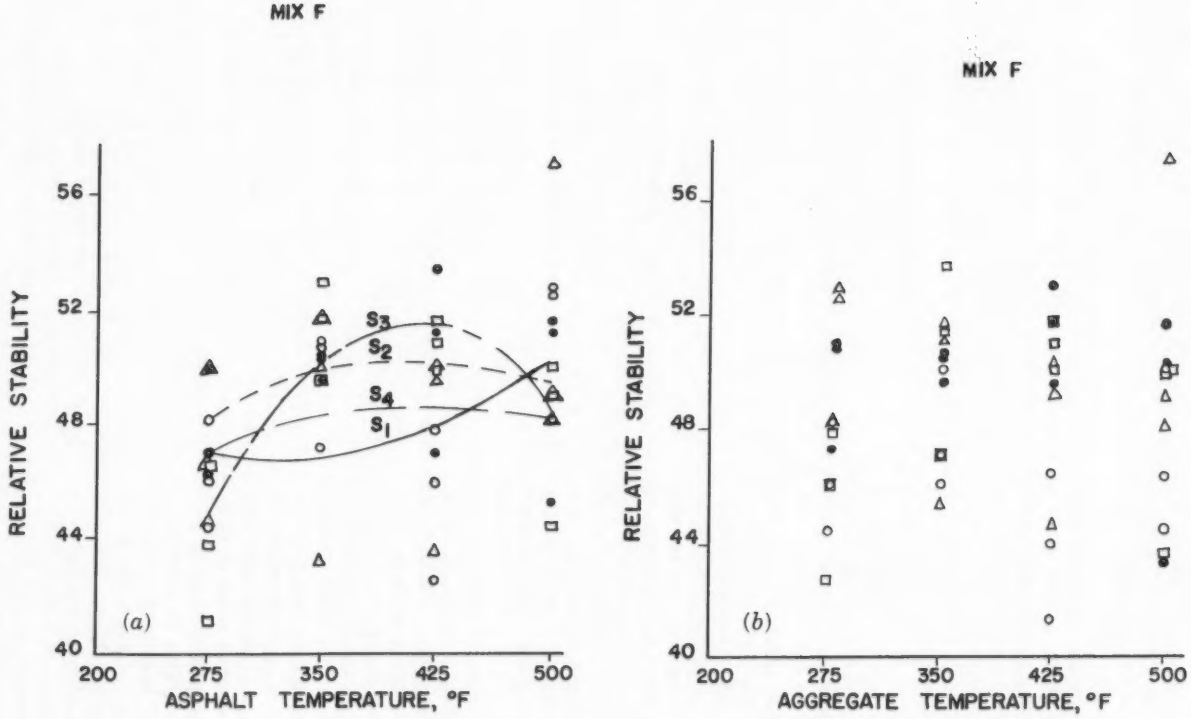

FIG. 6.—HVEEM STABILITY-TEMPERATURE RELATIONS

Current specifications[8] limit asphaltic concrete mix temperatures to 275-325° F. From relations shown in Table 3, there are indications of some hardening of the asphalt when heated to temperatures above 275° F. Whether these changes are significant in loss of durability cannot be determined without actual field tests.

An asphaltic concrete pavement must have stability and durability. A study of the observations made of the effect of high temperatures of asphalt or aggregate, or both, upon stability measures leads to the following conclusions. A higher stability will result if aggregate temperature is increased. However, a higher aggregate temperature during the mixing period will increase the temperature of the asphalt in this period. As asphalt temperature increases, there is a decrease in stability.

Physical characteristics measured by the tests employed in this study have already been mentioned. If all conclusions or suppositions are based on the Marshall test, it is seen that the overheating of the asphalt will have detrimental effects on the cohesive and viscous properties of the asphalt. The stability in the Marshall test is primarily a measure of these characteristics. However, as aggregate temperature is increased, there is some benefit gained. The binder will be more uniform throughout the mixture and compaction is facilitated. This will result in some degree of improvement in the mechanical stability of the mix. Since this study was undertaken to investigate effects of high temperatures on asphalt mixes the next step is to determine the point at which detrimental properties outweigh the benefits of higher mix temperatures. If one superimposes the stability-asphalt temperature relations on the stability-aggregate relations (see Fig. 1), this would result in a point of intersection at approximately 375° F. It is postulated that an increase in aggregate temperature during time of mixing will be of some value to the mix without seriously affecting the "life" of durability of the mix.

Before any definite statement is made to that effect, a study of other characteristics of the mixes should be made. In addition to a stability requirement in hot-mix design, there is a flow limitation in the Marshall test method. The coarser mix indicated little if any significant effect of asphalt or aggregate temperature on flow. In any case, the flow values in this investigation all remained below the maximum flow value of 20.[10]

The results on Mix F varied. The asphalt temperature-flow relation indicates a decrease followed by an increase in flow value. The aggregate temperature-flow relation indicates a steadily rising flow value. A study of the flow curves indicates that an aggregate temperature of 375° F could be used in asphaltic concrete mixing. The flow values ranged between 11 and 17 with a mean value of 13.5. The range of flow values shows that they fall below the maximum limit of 20 usually considered for highways.

If a study is made of the data appearing in Tables 5 and 6, it is seen that Mix C specimens contain 1.6% to 4.2% voids of total mix. The range of percentage of voids filled with bitumen is 74.9% to 89.3%, with the majority falling within 80% to 86%. Specimens of Mix F show a variation in percentage of voids of total mix between 6.2% and 9.7%, with the major portion falling within the range of 6.8% to 8.6%. The percentage of voids filled with bitumen ranges between 57.2% and 68.5%. The largest portion of the test specimens lies in the range of 60% to 65% of voids filled with bitumen.

From these observations, it can be seen that Mix C lies close to the minimum limit of percentage voids of total mix. There are some advantages to this condition. The surface would be impermeable to water and there would be

less chance for rapid oxidation or hardening of the asphalt in the mix. Also, an asphalt mixture containing the high penetration asphalt (85/100) and a high percentage of asphalt (6%) consistent with necessary stability should prove to be the most resistant to cracking at low temperatures. In some cases, a tolerance of 5% of aggregate voids filled with asphalt is permitted. This variation is not a general rule unless the asphalt-aggregate mix indicates satisfactory service behavior.[9]

Mix F lies at the other extreme. It lies below the suggested minimum percentage of voids filled with asphalt cement. Consequently, it can be said that such a surface course would harden and become brittle at a faster rate than would a surface course of Mix C. Due to the low percentage (60% to 65%) of voids filled with asphalt, there would be reduced resistance to cracking at low temperatures. Also, the aggregate for Mix F would be more costly to produce. Thus, it can be seen that although laboratory tests on Mix F indicate satisfactory stability measures and flow values, the final product for paving purposes would be very questionable from the standpoint of production and service behavior.

The Hveem stabilometer values for both mixes, and under temperature conditions used, proved to be satisfactory. The minimum relative stability value for unlimited traffic is thirty-five. There were only 3 cases out of 96 specimens where the relative stability value fell below 40. Even these values were above 37.

Thus, it can be stated that the effect of high temperature on the service behavior of the finished product cannot be determined by the stability measures used in this study. All the stability measures fell within the suggested limits. Use of the desired void-content does aid in selection or rejection of a final asphalt-aggregate combination. It was on this basis that Mix F was considered as having unsatisfactory properties for a final mix. The field performance of such a mix would be unsatisfactory.

It was stated earlier that the specifications limiting mix temperature could be changed. That is, raise the upper limit from 325° F to 375° F. By raising the temperature to which the aggregate can be heated, a better coating of the aggregate is obtained. Also, a mixture of desirable workability is available. The one question to be answered concerning this raising of the aggregate temperature is "Will this elevated temperature affect the durability of the finished pavement?"

An attempt will be made to answer this question. It must be emphasized here that the interpretations are based upon results of this laboratory study. From the results of this investigation and reports of other investigators, it will be shown that the temperature can be raised to 375° F.

Previous studies[4] indicate that plant mixing results in about a 20% loss in penetration. On the assumption that the asphalt was overheated en masse prior to mixing with aggregate at 375° F, the penetration could then be assumed to reach a value of 91. When this asphalt is sprayed into a pugmill with aggregate at 375° F its penetration value would drop to about 72. This then is assumed to represent the initial field condition of the asphalt-aggregate mix. By using the approach of Brown, Sparks, and Larsen,[13] it may be shown that an aggregate temperature of 375° F can be used without undesirable effects on pavement durability.

The asphalt used in this study was of 85/100 pen grade. The asphalt used in the Illinois tests[13] was of 60/70 pen grade. However, the mathematical relationship of change in penetration and aging should hold true for all asphalts,

following the hyperbolic relation developed.[13] The value of Y in this expression is the change of the property with time rather than the absolute magnitude of the property in question. The constants a and b depend upon the factors of road construction, traffic conditions, weathering conditions, and the intrinsic character of the asphalt.

During the construction of the pavements studied by Brown, Sparks, and Larsen[13] some of the hot-mix was exposed to temperatures as high as 375° F. By using the greater rate of change in penetration it is believed that the use of 375° F will be substantiated. The approximate percentage loss of penetration, based on initial field conditions, was as follows:

Time, years	1	3	5	7	9
Penetration Loss	31%	42%	50%	54%	54%

By using these percentages on the mixes studied in this experiment it is seen that the pavement would conceivably have the corresponding penetrations after being mixed at 375° F and then placed in the field.

Time, years	1	3	5	7	9
Penetration	50.7	41.7	36	33.1	33.1

To estimate the lowest value of penetration, use of the presumed penetration values is made. By using the relation established by Brown, Sparks, and Larsen,[13]

$$\Delta Y = \text{change in pen} = T/(a + b\,T) \quad \ldots\ldots\ldots\ldots \quad (3)$$

it is possible to evaluate the constants a and b having corresponding values ΔY and T (aging time, years). The writer found the numerical value of b to be 0.026. Brown, Sparks, and Larsen[13] report that the limiting change in penetration is defined by $y = 1/b$. Consequently, the limiting value of penetration would be approximately 33. This relationship would hold true as long as traffic and weather conditions remained the same over the years. It is believed that the greater the traffic the greater the protection against oxidation effects.[4,13] This would mean longer life with smaller losses of penetration than proposed previously.

Hubbard and Gollomb[12] report that pavements having penetration values below 30 tend to crack. That is, they show tendencies toward reduced resistance to contraction at low temperatures. In the hypothetical case described here, the limiting penetration is 33. This would indicate that the requirements imposed on the asphalt-aggregate mixes result in a pavement that would exhibit satisfactory service behavior. The hypothetical case is based on the worst possible conditions remaining the same. The hypothetical study leads the writer to conclude that an aggregate temperature of 375° F can be used satisfactorily when mixing an asphalt-aggregate combination for a highway surface course.

CONCLUSIONS

On the basis of this laboratory investigation, the following conclusions are indicated:

1. The Marshall stability of both gradations decreases as the asphalt preheat temperature increases.

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13 "Rate of Change of Softening Point, Penetration, and Ductility of Asphalt in Bituminous Pavement," by A. B. Brown, J. W. Sparks, and O. Larsen, Proceedings, The Assn. of Asphalt Paving Technologists, Vol. 26, 1957, p. 66.

2. The Marshall stability of both gradations increases as the aggregate temperature at the time of mixing and compaction increases.

3. The Hveem relative stability values at asphalt temperatures greater than 275° F are larger than the values obtained when asphalt preheated to 275° F is used in producing a mix.

4. The temperature of the aggregate at the time of mixing appears to have no significant effect on the Hveem relative stability.

5. It is satisfactory to use a temperature of 375° F when mixing asphalt and aggregate, provided that the Steinbaugh-Brown and Brown, Sparks, and Larsen criteria for change in rheological properties at the mix plant are valid, and that the Brown, Sparks, and Larsen hyperbolic relation of rheological property with time is a true relationship.

APPENDIX. SUMMARY OF TEST DATA AND ANALYSIS OF VARIANCE SUMMARY

Tables 5 through 9 (on the following pages) contain a summary of the test data and the variance analysis.

TABLE 5.—SUMMARY OF MARSHALL STABILITY TEST DATA

Spec. No.	Height (in.)	Volume (cc)	Spec. Grav.	Percent Voids		Stability, lb		Flow (0.01 in.)
				Total Mix	Filled W. AC	Measured	Convert	
M-1111	2.625	529.0	2.353	4.1	77.1	1513	1452	19
1112	2.578	524.7	2.380	3.0	82.3	1650	1584	18
1113	2.594	524.3	2.388	2.7	83.9	1550	1488	12
1211	2.594	526.0	2.387	2.7	83.9	1638	1572	18
1212	2.578	523.3	2.395	2.4	85.4	1663	1596	15
1213	2.500	504.1	2.403	2.1	87.0	1688	1756	18
1311	2.562	520.0	2.406	2.0	87.6	1820	1820	18
1312	2.562	521.9	2.402	2.1	87.0	1840	1840	16
1313	2.547	517.0	2.407	1.9	88.2	1967	1967	17
1411	2.594	526.1	2.399	2.2	86.5	2050	1968	16
1412	2.594	529.5	2.392	2.5	84.9	1967	1888	16
1413	2.625	524.7	2.402	2.1	87.0	1925	1848	14
2111	2.562	520.4	2.387	2.7	83.9	1480	1480	18
2112	2.594	527.1	2.372	3.3	80.8	1480	1421	17
2113	2.562	526.0	2.378	3.1	81.8	1588	1524	17
2211	2.594	526.8	2.393	2.5	84.9	1650	1584	16
2212	2.594	524.8	2.393	2.5	84.9	1713	1644	18
2213	2.578	524.2	2.392	2.5	84.9	1725	1656	18
2311	2.562	524.0	2.405	2.0	87.6	2050	1968	16
2312	2.578	524.7	2.399	2.3	86.0	1900	1824	14
2313	2.594	526.8	2.401	2.2	86.5	1840	1766	16
2411	2.562	525.6	2.403	2.1	87.1	1967	1888	18
2412	2.562	526.3	2.390	2.6	84.4	1886	1811	17
2413	2.562	526.3	2.387	2.7	83.9	1950	1872	16
3111	2.594	527.9	2.384	2.9	82.9	1575	1512	17
3112	2.578	525.2	2.392	2.5	84.9	1600	1536	17
3113	2.594	524.3	2.394	2.4	85.4	1625	1560	16
3211	2.594	531.8	2.378	3.1	81.8	1638	1572	16
3212	2.562	525.6	2.397	2.3	86.0	1650	1584	16

3213	2,578	527.7	2,388	2.7	83.9	1675	1608	15
3311	2,594	526.3	2,401	2.2	86.5	1871	1796	15
3312	2,594	525.8	2,395	2.4	85.4	1738	1668	17
3313	2,594	527.1	2,390	2.6	84.4	1738	1668	15
3411	2,578	524.5	2,404	2.0	87.6	1800	1728	18
3412	2,562	525.3	2,402	2.1	87.1	1984	1905	17
3413	2,562	523.2	2,403	2.1	87.1	1886	1811	15
4111	2,609	532.3	2,365	3.6	79.4	1480	1421	15
4112	2,625	534.5	2,361	3.8	78.5	1460	1402	15
4113	2,625	532.9	2,366	3.6	79.4	1500	1440	19
4211	2,547	518.5	2,394	2.4	85.4	1650	1650	17
4212	2,609	528.7	2,392	2.5	84.9	1650	1584	17
4213	2,594	529.5	2,388	2.7	83.9	1688	1620	18
4311	2,625	529.0	2,390	2.6	84.4	1738	1668	15
4312	2,578	525.7	2,396	2.4	85.4	1857	1783	17
4313	2,609	529.3	2,383	2.9	82.8	1700	1632	16
4411	2,578	522.9	2,410	1.8	88.7	1886	1811	18
4412	2,578	524.0	2,403	2.1	87.1	1840	1766	20
4413	2,609	524.2	2,395	2.4	85.4	1871	1796	15
1121	2,750	562.5	2,249	8.1	62.0	1725	1484	14
1122	2,312	473.0	2,273	7.1	65.3	1525	1738	12
1123	2,312	470.8	2,251	8.0	62.3	1429	1629	12
1221	2,500	506.7	2,271	7.2	65.0	1700	1768	14
1222	2,500	509.5	2,261	7.6	63.6	1775	1775	17
1223	2,469	504.2	2,257	7.7	63.3	1925	2002	15
1321	2,438	502.4	2,280	6.8	66.3	1886	1961	13
1322	2,531	520.5	2,276	7.0	65.6	2084	2084	14
1323	2,469	503.2	2,277	6.9	66.0	2140	2226	14
1421	2,500	509.2	2,277	6.9	66.0	1967	1967	15
1422	2,484	505.7	2,278	6.9	66.0	2025	2106	13
1423	2,484	508.7	2,255	7.8	63.1	2160	2160	14
2121	2,453	505.3	2,241	8.4	61.1	1588	1652	12
2122	2,531	521.1	2,249	8.1	62.0	1663	1663	12
2123	2,594	537.0	2,230	8.8	59.8	1725	1604	12
2221	2,547	522.6	2,257	7.7	63.3	1738	1668	14
2222	2,531	516.2	2,256	7.8	63.0	1725	1725	11
2223	2,469	506.5	2,246	8.2	61.7	1775	1846	13
2321	2,453	505.2	2,264	7.4	64.3	1713	1782	12
2322	2,469	507.6	2,266	7.4	64.3	1750	1820	14

TABLE 5.—SUMMARY OF MARSHALL STABILITY TEST DATA (CONT'D)

Spec. No.	Height (in.)	Volume (cc)	Spec. Grav.	Percent Voids		Stability, lb		Flow (0.01 in.)
				Total Mix	Filled W. AC	Measured	Convert	
M-2323	2.516	517.1	2.246	8.2	61.7	1871	1871	14
2421	2.531	519.1	2.274	7.0	65.6	1983	1983	13
2422	2.500	512.1	2.275	7.0	65.6	2050	2050	14
2423	2.484	510.1	2.265	7.4	64.3	2250	2250	14
3121	2.562	527.0	2.239	8.5	60.8	1625	1560	12
3122	2.562	526.4	2.238	8.5	60.7	1575	1512	11
3123	2.531	518.4	2.231	8.8	59.8	1563	1563	13
3221	2.609	537.6	2.254	7.8	62.9	1775	1651	14
3222	2.594	533.9	2.260	7.6	63.6	1800	1728	14
3223	2.594	531.1	2.248	8.1	62.0	1857	1783	13
3321	2.594	532.1	2.262	7.5	63.9	1775	1704	12
3322	2.594	536.3	2.260	7.6	63.6	2200	2046	15
3323	2.609	537.0	2.246	8.2	61.7	2200	2046	14
3421	2.641	543.6	2.270	7.2	64.9	2238	2081	15
3422	2.625	542.5	2.260	7.6	63.6	2306	2145	12
3423	2.625	542.2	2.256	7.8	63.0	2360	2195	14
4121	2.594	532.4	2.234	8.7	60.1	1550	1488	13
4122	2.625	540.6	2.230	8.8	59.8	1675	1558	14
4123	2.625	544.2	2.209	9.7	57.2	1650	1534	12
4221	2.672	548.9	2.248	8.1	62.0	1800	1602	14
4222	2.656	546.2	2.252	7.9	62.6	1820	1693	13
4223	2.672	548.6	2.237	8.5	60.7	1857	1653	13
4321	2.609	535.4	2.248	8.1	62.0	2050	1968	14
4322	2.625	534.6	2.255	7.8	62.9	2140	2054	13
4323	2.656	539.5	2.232	8.7	60.1	2025	1883	15
4421	2.641	537.9	2.294	6.2	68.5	2450	2278	17
4422	2.641	536.0	2.291	6.3	68.1	2434	2264	15
4423	2.625	536.4	2.275	7.0	65.6	2250	2092	15

TABLE 6.—SUMMARY OF HVEEN STABILOMETER TEST DATA

Spec. No.	Height (in)	Spec. Grav.	Percent Voids		Hveem Gauge		Vert. Load lbs.	Final Displ. (in.)	Percent Stability	Corr. Stability
			Total Mix	Filled W. AC	psia	psib				
M-1114	2.578	2.391	2.6	84.4	49.0	171.0	11,000	0.263	37.7	37.7
1115	2.594	2.400	2.2	86.5	38.0	172.0	11,000	0.287	42.5	42.5
1116	2.578	2.410	1.8	88.7	37.0	178.0	11,000	0.274	44.4	44.4
1214	2.563	2.410	1.8	88.7	38.0	164.0	11,000	0.239	46.7	46.7
1215	2.562	2.396	2.4	85.4	42.0	173.0	11,000	0.252	43.0	43.0
1216	2.562	2.399	2.2	86.5	41.0	170.0	11,000	0.233	45.4	45.4
1314	2.547	2.401	2.2	86.4	44.0	167.0	11,000	0.236	43.1	43.1
1315	2.562	2.398	2.3	86.0	45.0	171.0	11,000	0.254	40.7	40.7
1316	2.562	2.402	2.1	87.0	39.0	163.0	11,000	0.252	44.9	44.9
1414	2.547	2.412	1.7	89.3	35.0	174.0	12,000	0.233	50.0	50.0
1415	2.547	2.413	1.7	89.3	46.0	171.0	11,000	0.236	41.8	41.8
1416	2.562	2.407	1.9	88.2	43.0	182.0	12,000	0.230	44.5	44.5
2114	2.562	2.391	2.6	84.4	45.0	184.0	11,000	0.236	42.5	42.5
2115	2.594	2.394	2.4	85.4	50.0	189.0	11,000	0.228	40.4	40.4
2116	2.594	2.392	2.5	84.9	42.0	172.0	11,000	0.227	45.3	45.3
2214	2.578	2.396	2.4	85.4	43.5	169.5	11,000	0.236	43.5	43.5
2215	2.594	2.396	2.4	85.4	42.5	170.5	11,000	0.242	43.5	43.5
2216	2.578	2.397	2.3	86.0	48.5	179.5	11,000	0.246	39.5	39.5
2314	2.594	2.398	2.3	86.0	48.5	176.5	11,000	0.224	41.6	41.6
2315	2.578	2.397	2.3	86.0	46.5	174.5	11,000	0.237	41.4	41.4
2316	2.594	2.392	2.5	84.9	48.5	176.5	11,000	0.229	41.0	41.0
2414	2.562	2.408	1.9	88.2	55.5	185.5	11,000	0.219	38.5	38.5
2415	2.562	2.401	2.2	86.5	46.0	166.0	11,000	0.218	43.9	43.9
2416	2.562	2.401	2.2	86.5	50.5	172.5	11,000	0.217	41.2	41.2
3114	2.594	2.380	3.0	82.3	44.5	189.5	11,000	0.230	43.5	43.5
3115	2.594	2.381	3.0	82.3	44.5	184.5	11,000	0.218	44.8	44.8
3116	2.594	2.391	2.6	84.4	50.0	186.0	11,000	0.212	42.0	42.0
3214	2.594	2.397	2.3	86.0	41.5	194.5	12,000	0.240	44.5	44.5
3215	2.594	2.404	2.0	87.6	46.5	194.5	11,800	0.223	43.0	43.0
3216	2.594	2.391	2.6	84.4	43.0	194.0	12,000	0.222	45.2	45.2
3314	2.562	2.399	2.2	86.5	43.5	194.5	12,000	0.210	46.1	46.1
3315	2.562	2.408	1.9	88.2	41.5	185.5	12,000	0.204	48.3	48.3
3316	2.578	2.393	2.5	84.9	51.5	194.5	11,900	0.208	41.7	41.7
3414	2.578	2.402	2.1	87.1	56.5	189.5	11,000	0.184	42.5	42.5

TABLE 6.—SUMMARY OF HVEEM STABILOMETER TEST DATA (CONT'D)

Spec. No.	Height (in)	Spec. Grav.	Percent Voids		Hveem Gauge		Vert. Load lbs.	Final Displ. (in.)	Percent Stability	Corr. Stability
			Total Mix	Filled W. AC	psi[a]	psi[b]				
M-3415	2.562	2.405	2.0	87.6	44.0	194.0	12,050	0.204	46.5	46.5
3416	2.578	2.390	2.6	84.4	48.5	194.5	11,750	0.196	45.0	45.0
4114	2.609	2.374	3.3	80.9	41.5	194.5	11,575	0.235	44.9	48.2
4115	2.594	2.396	2.4	85.4	43.5	194.5	11,500	0.206	46.7	46.7
4116	2.609	2.364	3.7	79.0	45.5	188.5	11,000	0.224	43.6	47.0
4214	2.609	2.391	2.6	84.4	52.5	194.5	11,420	0.217	40.2	43.8
4215	2.594	2.401	2.2	86.5	54.5	194.5	11,340	0.189	42.8	42.8
4216	2.625	2.378	3.1	81.8	43.5	194.5	11,960	0.211	46.0	49.2
4314	2.609	2.399	2.2	86.5	52.5	194.5	11,760	0.227	39.1	42.5
4315	2.578	2.405	2.0	87.6	48.0	192.0	11,600	0.197	45.1	45.1
4316	2.594	2.388	2.7	83.9	50.5	194.5	11,460	0.202	43.0	43.0
4414	2.594	2.404	2.0	87.6	57.5	194.5	11,420	0.188	41.0	41.0
4415	2.609	2.407	1.9	88.2	45.0	179.0	11,760	0.214	45.0	48.2
4416	2.594	2.408	1.9	88.2	50.0	194.0	11,680	0.192	44.8	44.8
1124	2.578	2.231	8.8	59.6	42.0	180.0	10,000	0.237	44.4	44.4
1125	2.562	2.256	7.8	63.0	40.0	161.0	10,000	0.233	46.0	46.0
1126	2.500	2.249	8.1	62.0	38.0	165.0	10,000	0.226	48.2	48.2
1224	2.562	2.237	8.5	60.7	39.0	175.0	11,000	0.206	50.0	50.0
1225	2.562	2.229	8.9	59.5	44.0	189.0	11,000	0.209	46.0	46.0
1226	2.562	2.249	8.1	62.0	40.0	193.0	11,000	0.224	47.0	47.0
1324	2.594	2.282	6.7	66.7	42.0	180.0	11,000	0.215	46.5	46.5
1325	2.500	2.277	6.9	66.0	51.0	174.0	10,000	0.213	41.3	41.3
1326	2.375	2.275	7.0	65.6	40.0	166.0	11,000	0.214	48.0	44.0
1424	2.500	2.278	6.9	63.0	45.0	169.0	11,000	0.201	46.3	46.3
1425	2.562	2.279	6.8	66.3	42.0	168.0	11,000	0.191	50.0	50.0
1426	2.500	2.267	7.3	64.6	44.0	186.0	12,000	0.224	44.5	44.5
2124	2.594	2.228	8.9	59.5	44.0	180.0	10,000	0.201	47.2	47.2
2125	2.438	2.251	8.0	62.3	38.5	181.5	11,000	0.202	50.7	50.7
2126	2.469	2.239	8.5	60.8	38.5	178.5	11,000	0.200	50.9	50.9
2224	2.469	2.266	7.4	64.3	39.5	178.5	11,000	0.200	50.4	50.4
2225	2.562	2.265	7.4	64.3	40.5	190.5	11,000	0.195	50.5	50.5
2226	2.438	2.266	7.4	64.3	42.5	169.5	11,000	0.190	49.5	49.5
2324	2.438	2.278	6.9	66.0	42.5	177.5	11,000	0.190	49.5	49.5
2325	2.406	2.274	7.0	65.6	36.5	187.5	12,000	0.193	53.0	53.0

2326	2.469	2.252	7.9	62.6	38.5	186.5	12,000	0.196	51.7	51.7
2424	2.594	2.266	7.4	64.3	52.5	188.5	11,000	0.193	43.3	43.3
2425	2.625	2.257	7.7	63.3	45.5	174.5	11,000	0.184	48.6	51.6
2426	2.531	2.274	7.0	65.6	42.5	186.5	11,000	0.186	50.1	50.1
3124	2.594	2.247	8.1	62.0	52.0	177.0	10,000	0.198	42.6	42.6
3125	2.594	2.240	8.4	61.3	50.0	194.0	10,800	0.182	46.0	46.0
3126	2.594	2.243	8.3	61.4	47.5	183.5	10,000	0.180	47.8	47.8
3224	2.594	2.258	7.7	63.3	48.0	194.0	10,900	0.184	47.0	47.0
3225	2.562	2.262	7.5	63.9	40.5	188.5	11,000	0.187	51.3	51.3
3226	2.594	2.239	8.5	60.8	39.0	194.0	11,550	0.178	53.6	53.6
3324	2.609	2.263	7.5	63.9	48.5	190.5	11,000	0.184	46.7	50.0
3325	2.656	2.254	7.8	62.9	44.0	194.0	11,850	0.188	48.8	51.7
3326	2.640	2.256	7.8	63.0	48.5	189.5	11,000	0.174	48.2	51.0
3424	2.625	2.268	7.2	64.9	60.0	194.0	10,550	0.188	40.0	43.6
3425	2.625	2.251	8.0	62.3	47.5	194.5	11,350	0.189	46.7	50.0
3426	2.578	2.284	6.6	67.0	46.5	194.5	11,300	0.171	50.0	50.0
4124	2.609	2.236	8.6	60.4	51.5	184.5	10,000	0.183	45.0	48.2
4125	2.625	2.225	9.0	59.2	46.5	194.5	10,650	0.171	49.8	52.5
4126	2.531	2.228	8.9	59.5	40.5	194.5	10,900	0.181	52.8	52.8
4224	2.594	2.294	6.2	68.5	43.5	194.5	10,420	0.172	45.3	45.3
4225	2.656	2.244	8.3	61.4	44.5	194.5	11,000	0.189	48.3	51.2
4226	2.625	2.265	7.4	64.3	42.5	194.5	11,280	0.195	48.8	51.5
4324	2.625	2.260	7.6	63.6	63.5	194.5	10,460	0.168	41.0	44.6
4325	2.640	2.238	8.5	60.7	48.0	194.0	11,400	0.177	47.3	50.2
4326	2.625	2.263	7.5	63.9	48.5	194.5	11,340	0.187	46.0	49.2
4424	2.688	2.273	7.1	65.3	53.0	194.0	10,800	0.172	45.8	49.1
4425	2.719	2.247	8.1	62.0	54.0	193.0	11,000	0.181	44.0	47.3
4426	2.672	2.273	7.1	65.3	54.0	186.0	11,000	0.176	44.8	48.1

[a] At vertical load of 5,000 lbs.
[b] At vertical load shown in table.

TABLE 7.—ANALYSIS OF VARIANCE, MARSHALL STABILITY, MIX C

Source of Variation (1)	Sum of Squares (2)	D.F. (3)	Mean Squares (4)	F (5)	$F_{0.05}$ (6)
Asphalt Temp (A)	70,629.8	3	23,543.3	5.90[a]	2.90
Aggregate Temp (S)	940,181.6	3	313,393.9	78.60[a]	2.90
Interaction: AxS	56,843.9	9	6,316.0	1.58	2.19
Subtotal	1,067,655.3	15			
Error	127,584.7	32	3,987.0		
Total	1,195,240.0	47			

[a] Indicates significance.

TABLE 8.—COMPONENTS OF THE MAIN EFFECTS

Specific Component (1)	Sum of Squares (2)	D.F. (3)	Mean Square (4)	F (5)	$F_{0.05}$ (6)
Linear Comp (A_L)	70,315.3	1	70,315.3	17.64[a]	4.15
Quadratic Comp (A_Q)	24.1	1	24.1	0.01	4.15
Cubic Comp (A_C)	290.4	1	290.4	0.07	4.15
Linear Comp (S_L)	911,433.8	1	911,433.8	228.60[a]	4.15
Quadratic Comp (S_Q)	17,404.1	1	17,404.1	4.37[a]	4.15
Cubic Comp (S_C)	11,343.8	1	11,343.8	2.85	4.15
Error	127,584.7	32	3,987.0		

[a] Indicates significance.

TABLE 9.—COMPONENTS OF THE INTERACTION

Specific Component (1)	Sum of Squares (2)	D.F. (3)	Mean Squares (4)	F (5)	$F_{0.05}$ (6)
$S_L A_L$	9,009.12	1			
$S_Q A_L$	1,859.27	1			
$S_C A_L$	18,127,41	1			
Total	28,995.80	3	9,665.27	2.42	2.90
$S_L A_Q$	2,760.82	1			
$S_Q A_Q$	2,324.08	1			
$S_C A_Q$	1,696.02	1			
Total	6,780.92	3	2,260.30	0.57	2.90
$S_L A_C$	7,620.48	1			
$S_Q A_C$	13,261.07	1			
$S_C A_C$	185.65	1			
Total	21,067.20	3	7,022.40	1.76	2.90
$A_L S_L$	9,009.12	1			
$A_Q S_L$	2,760.81	1			
$A_C S_L$	7,620,48	1			
Total	19,390.41	3	6,463.47	1.62	2.90
$A_L S_Q$	1,859.26	1			
$A_Q S_Q$	2,324.08	1			
$A_C S_Q$	13,261.07	1			
Total	17,444.41	3	5,814.80	1.46	2.90
$A_L S_C$	18,127.41	1			
$A_Q S_C$	1,696.02	1			
$A_C S_C$	185.65	1			
Total	20,009.08	3	6,669.69	1.67	2.90
Error	127,584.70	32	3,987.02		

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FREEWAY SPACING IN AN URBAN FREEWAY SYSTEM

By James M. Peterson,[1] M. ASCE

SYNOPSIS

The various controls that influence freeway spacing in an urban freeway system are described and categorized. A simplified study procedure is outlined that illustrates the interrelationships of certain variables affecting freeway spacing in a grid system.

It is estimated that by 1975, two-thirds of our rapidly mushrooming population will be concentrated in less than twenty super-urban regions.[2] Attending this growth and concentration of population will be great changes in governmental administration, social and economic activities, land use planning and communication. Of all the forces shaping these changes, the most important are those based on transportation needs. To the planning engineer has fallen the task of providing a circulation system that will adequately handle the travel linkages between the various land uses for today's needs and those of the foreseeable future.

To adequately provide for the mounting volume of vehicular traffic, any large urban area requires some form of a freeway system, be it rectangular, radiating, circumferential, or a combination of these. It is axiomatic to say that this system should be well integrated with the local street system in order to provide optimum traffic service. Regardless of the configuration of the system or the spacing of the freeways within it, these freeways will fill as the years pass. However, as the freeway system develops, the planning engineer

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[2] U. S. News and World Report, November, 28, 1958, Interview with Dr. Philip M. Hauser.

is constantly concerned with whether or not the individual freeways are properly spaced to provide optimum traffic service and also permit orderly community development. The amount of traffic that will be diverted to a freeway can be predicted by expressing percent users as a function of time and distance savings or freeway-city street time ratios. This is just one of the factors required to solve the complex problem of freeway spacing in an urban freeway system.

In recent years, there has been an increasing number of cities that have based their freeway-system needs on area wide transportation studies. These studies include the assignment to a system of future traffic based on population distribution, land use pattern, trip generation, and interzonal traffic volumes. In the Chicago Study and elsewhere a great deal of work is being done to develop computer programs that take into account all the variables affecting travel in an urban area, including such factors as motor vehicle ownership, trip length distribution, employment, population distribution, land use, trip purpose, and travel times. Analyses of this type permit systematic adjustment and retesting until the freeway system exhibits maximum traffic-service characteristics. It seems appropriate, before embarking on such an extensive program, to develop an appreciation of the various basic controls involved in freeway spacing in an urban freeway system.

Physical controls on freeway spacing are those imposed by the distance requirements for geometrics and signing. These are the distances necessary to develop reasonable grades, permit safe merging and weaving, and allow the motorist sufficient recognition and decision time at interchanges and ramps.

The distances shown in Fig. 1 are desirable minimums and result in a spacing between freeways of about 2 miles. Note that this spacing allows only one full set of ramps from the city streets connecting to the freeway between freeway interchanges. As a rule it will be necessary to increase this spacing 1 mile for each additional full set of ramps. Thus a freeway system based on 4-mile spacing could provide three complete on and off movements from the freeway to the city streets. Fig. 1 is also based on the desirable design practice of having no ramps to the city streets within the area of an interchange between two freeways.

The three cases shown in Fig. 1 reflect typical ramp patterns for handling design peak hour traffic desires. Case 1 is a four quadrant cloverleaf with 200-ft-radius loops and a system of collector-distributor roads. It is assumed that the collector-distributor road enters the freeway as a single lane on-ramp. This limits the design peak-hour volume entering the freeway from one direction to about 1,500 vehicles (the capacity of one freeway lane). This is approximately the number of vehicles that could be expected to come from a city street that carries, on the average, about 500 vehicles per hour per lane.

Case 2 illustrates a typical full diamond that would be used in locations where the design peak-hour-volume entering the freeway in one direction does not exceed 1,000 vehicles per hr. Case 3 shows a method for doubling the number of connections to the city streets without increasing the number of connections to the freeway over four. A design such as this would be required in an area where the individual street capacities are not high enough to permit picking up the traffic at one point such as in Cases 1 and 2. The design peak hour volume entering the freeway in one direction for Case 3 would be 1,500 vehicles. Again, this is the practical capacity of a single freeway lane and represents about all the vehicles that could be drawn from two average city streets.

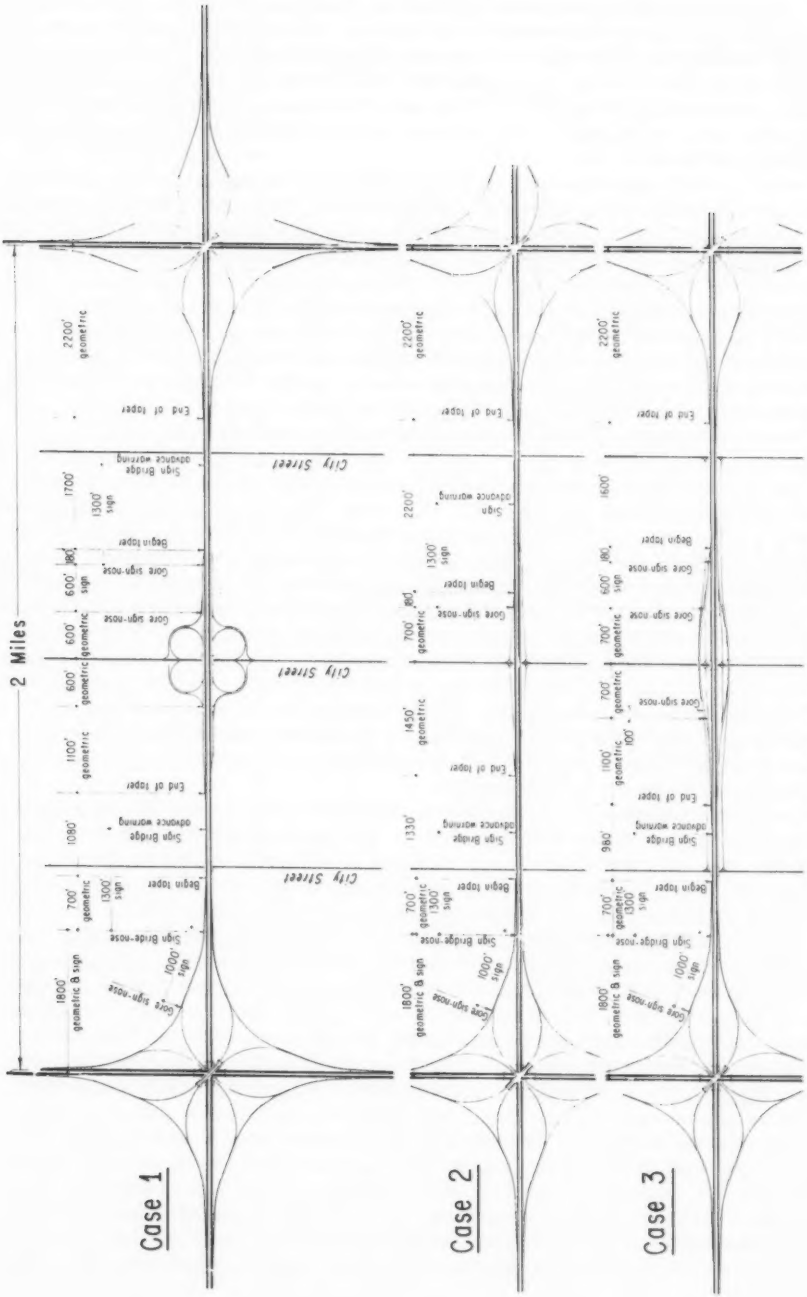

FIG. 1.—FREEWAY SPACING PHYSICAL CONTROLS

The existing master-planned city street pattern will control freeway spacing to some degree. The city street pattern affects the type and number of ramp connections which, in turn, affect freeway spacing. Of the practical controls on freeway spacing most important is probably the availability of funds in the planned future for the freeway system. There is no point in beginning construction, or even planning for a freeway system that cannot be financed in the foreseeable future.

Another very practical consideration affecting freeway spacing is the variability of traffic desire found in any urban area. This variability is directly related to the intensity of use of the main categories of land, residential, commerical and industrial. Trip generation density is dependent on the type and intensity of land use. Hence, highly developed areas will generate the most trips and require closer spacing of freeways than areas of less growth.

Topography often imposes definite controls on a freeway system. Location problems caused by bays, peninsulas, and hills are well known. Circumvention of these natural barriers can usually be justified because of excessive construction costs and may often be justified on the basis of traffic service.

An important practical control from an operational standpoint is the balance between freeway and city street service. All trips result from the travel desires between land uses. The basic function of the freeway is to provide for most of the movement portion of the trip. The city street permits fulfillment of the trip purpose by providing terminal distribution of traffic and direct access to the land. The city streets also serve the short trips. If freeways and ramp connections are too close together, many of the short trips are attracted to the freeway. The resultant congestion impairs the freeway's function of movement. If freeways and connections are too far apart, too many intermediate length trips will remain on the city street system, causing congestion and delay. The proper balance of freeways and city streets permits each system to supplement the other and operate at optimum efficiency. The optimum condition represents a delicate and often not too clearly defined balance between trip length distribution, trip generation density, trip purpose, and capacities of city streets, ramps, and freeways.

In recent studies made in California for the establishment of a state-wide system of freeways and expressways, it was determined that the completed freeway system in the greater Los Angeles area would carry approximately 52% of the vehicle miles. This is in contrast to the more rural areas of the state, in which the freeway system is expected to carry about 70%.

There are many intangible controls on freeway spacing which assume real significance when developing a freeway system that will receive the backing of both the general public and local officials. These controls are termed intangible because they are difficult to express as concrete facts and figures. They are probably best described as community benefits. Loss of tax base falls in the preceding category. Unquestionably the more right of way a freeway pattern requires, the more land is removed from the local tax rolls. Local communities are understandably concerned with possible loss of tax base. However, it is considered that this loss of tax base is principally an imaginary one. Surveys conducted in California do not indicate a reverse trend in over-all tax base even in small communities where millions of dollars worth of right of way have been purchased for freeway use. Furthermore, it has generally proved true that over the years land values appreciate near a freeway system. This is particularly true in areas that can be developed for commercial or industrial use.

Removal of productive capacity caused by right of way acquisition is another undesirable immediate affect of the freeway. However, obviating the need for a freeway by excessive removal of trip producing land is quite unlikely. While the removal of commercial, industrial, or residential land may be undesirable in the short run, it is usually compensated for by more intensive use of the land adjacent to the freeway.

The economy of a freeway creates an intangible control on freeway spacing. It is a generally accepted tenet that the money invested in construction of a freeway must develop over a period of years a return to the highway user who pays for it. This return is usually termed user savings.

The computation of highway user savings is based on the value of savings that will accrue to the motorist (usually over an amortization period of 20 yr) in reduced travel time, and reduced operating costs by utilizing the proposed freeway facility in lieu of existing surface streets. Monetary values have been developed for computing user savings. For automobiles freeway operation is often rated at 4.5¢ per mile as compared to 4.75¢ per mile on conventional highways and city streets. The figure 2.6¢ is often assumed to be the value of a passenger-vehicle minute. The difficulty in properly defining the worth of short increments of time saved over a 20 yr period leads us to place user savings as a whole in the intangible category. User savings are computed for various freeway location lines, and the best line, other factors being equal, is usually considered to be the one with the largest user savings. Thus, in a sense, user savings may have considerable influence on the location of a freeway.

Comfort and convenience are intangibles that influence a freeway system. The extent to which the public will support a freeway program is a measure of how much the motorist is willing to pay for comfort and convenience of travel. There is little doubt that the motorist is willing to pay, but how much is difficult to evaluate.

From a traffic service standpoint the ideal approach to the freeway spacing problem would be to test and retest various configurations of freeways for an entire urban area using the latest parameters of travel and the most recent computer programs. Mention of such an undertaking is beyond the scope of this paper. However, on a theoretical basis, it is possible to show the relationships between freeway spacing, freeway access spacing, trip length distribution, and vehicle speeds. It is also possible to derive the relative traffic service provided by the freeway system and the city street system. An analysis of this type using various freeway and freeway access spacings has been partially completed (as of 1960) by the California Division of Highways. A general description of the study procedures and the results based on 4 mile spacing of freeways will serve to illustrate the relationships previously mentioned.

The problem is to compute the number of automobile trips generated in a selected area and determine the number that would use a grid-type freeway system with 4 miles between freeways.

Solution and Assumptions.—

Step 1.—Select a 16 sq mile area and assume that the freeways running N-S and E-W cross at the center of the area, dividing it into NE, SE, SW, and NW quadrants.

Step 2.—Assume a spacing for access points from the city streets to the freeways with no access points at the center of the area where the two freeways interchange.

Step 3.—Assume or develop constants for (a) freeway and city street speeds, (b) population of the area, (c) land use of the area, (d) number of dwelling units, (e) average daily residential trip origins per dwelling unit, (f) average daily vehicle trip origins per net acre of commercial land, (g) same as (f) for industrial land, (h) overall trip length distribution (airline), and (i) time-distance assignment curves to determine percent of freeway users for each trip length category.

Step 4.—From these constants, compute the number of vehicle trip origins in the 16 sq miles and assume they are evenly distributed over the area and that desire for travel is equal in each octant of a circle.

Step 5.—Assume that the trip origins are concentrated in a number of zone centroids.

Step 6.—Analyze the zones in one quadrant and apply the results to the other three quadrants by identity.

Step 7.—Develop percentage freeway assignment curves for parallel and diagonal users. (Parallel includes all directions from 22-1/2° on one side to 22-1/2° on the other side of N,S,E, or W. Diagonal includes all directions from 22-1/2° on one side to 22-1/2° on the other side of NE,SE,SW, or NW).

Step 8.—With the percentage freeway assignment curves assign the freeway users (Parallel and diagonal) from each quadrant to the proper freeway access points.

Step 9.—Compute the one-way annual daily traffic (ADT) on one freeway by totaling the assigned trips at successive access points. Four times this figure is the total one-way ADT leaving a 16 sq mile area.

The example to follow demonstrates the procedure outlined. All of the computations for this example were performed manually, but it should be noted that the entire process could be programmed for high speed computers utilizing, in part, the "shortest path through a maze," technique.[3] Storage of assignment curves and actual assignment to a network has also been programmed for computers.

EXAMPLE

Step 1.—The general area selected is an incorporated suburb in the greater urban area, chosen for its general homogeneity and location on comparatively level terrain. The area is, for the most part, composed of middle class residential neighborhoods with typical commercial development. There is no industrial land use. The N-S, E-W freeways were assumed to cross in the center of the 16 sq mile study area.

Step 2.—Access points were located at mile intervals (Fig. 2). The street pattern was assumed to be a grid, parallel to the freeway grid, with major streets at mile intervals and secondary streets on the 1/4 mile and 1/2 mile points. It was assumed that this system of streets and freeways extended, without limit, in all directions.

Step 3.—(a) Assumed speeds were 52 mph on the freeways and 25 mph on the city streets. (b) The population data were obtained from the local county Regional Planning Commission and averaged 8,859 persons per sq mile. (c) The commercial land use was determined on the basis of 2.6 net acres of com-

[3] "The shortest path through a maze," by Edward F. Moore, paper presented at the Internatl. Symposium on the Theory of Switching, Harvard Univ., Cambridge, Mass., 1957.

mercial land need for each 1,000 persons. There was no industrial land. (d) The number of dwelling units was obtained by assuming three persons per dwelling unit and dividing this into the 8,859 persons per sq mile giving 2,953 dwelling units. (e) Daily residential vehicle trip origins per dwelling unit were taken as four, based on generation studies by the local county road department. Residential trip origins per square mile then equals (2,953) (4) =

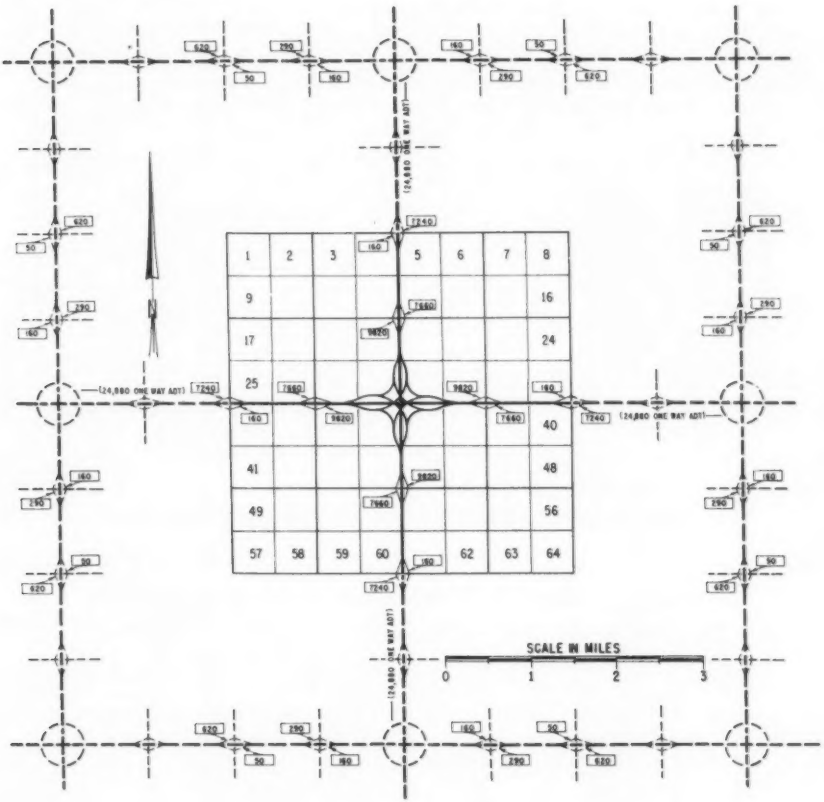

FIG. 2.—THEORETICAL TRAFFIC GENERATION FROM A 16 SQ. MILE AREA

11,812. (f) The average daily vehicle trip origins per net acre of commercial land were taken as 250. This is comparable to figures developed in San Diego, California and Detroit, Michigan. The total commercial origins per square mile is $\left(\frac{8,859}{1,000}\right)$ (2.6) (250) = 5,758. (g) No industrial origins were used in this residential area. (h) Percentage trip length distribution (airline) was taken from a motor vehicle use study conducted by the California Division

of Highways for the local urban area. (i) the "Percent of Traffic Diversion to Freeway" curves (Fig. 3) used by the California Division of Highways were used in comparing the city street trips with freeway trips.

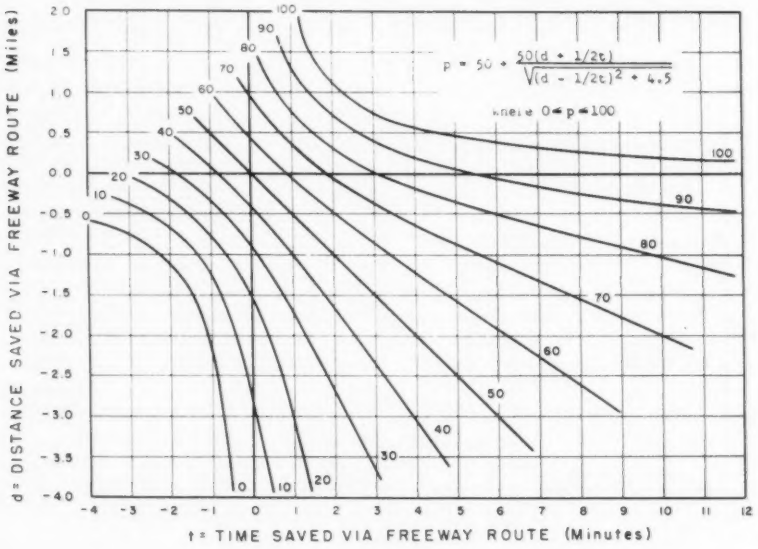

FIG. 3.—PERCENT OF TRAFFIC DIVERSION TO FREEWAY

TABLE 1.—PERCENTAGE DISTRIBUTION OF TRIPS (AIRLINE)

Mile	Percent	Mile	Percent	Mile	Percent
.5	10.29	11	1.93	22	0.35
1	20.55	12	1.63	23	0.31
2	14.00	13	1.39	24	0.27
3	9.98	14	1.17	25	0.23
4	7.54	15	1.00	26	0.22
5	5.97	16	0.86	27	0.19
6	4.80	17	0.72	28	0.17
7	3.92	18	0.64	29	0.15
8	3.28	19	0.54	30	0.13
9	2.74	20	0.47	> 30	1.87
10	2.28	21	0.41		

Step 4.—The total daily vehicle trip origins per square mile was 11,812 + 5,758 = 17,570. This figure was rounded to 18,000 for the actual assignment computations. The 16 sq. mile area generated a total of 288,000 trip origins. The assumption of even distribution and equal travel desire was made.

Step 5.—The 16 square miles were divided into 64 quarter square mile zones (Fig. 2). This zone size was chosen to minimize centroid bias and still keep the number of calculations within the limit of manual operation.

Step 6.—Since it was assumed that the origins were evenly distributed and there was equal desire for travel in all directions, only the zones in one quadrant were analyzed. These zones, shown in Fig. 4, were divided into three classifications according to their distances from the freeway. In other words, 43.75% of the zones were in the tier of zones immediately adjacent to the freeway, 31.25% were in the next tier away, and 25% were two tiers away. In this case, the NE quadrant was analyzed and the results applied to the other three quadrants by identity.

Step 7.—The percentage freeway assignment (Fig. 4) curves for parallel and diagonal users for the three classifications of zones were developed for

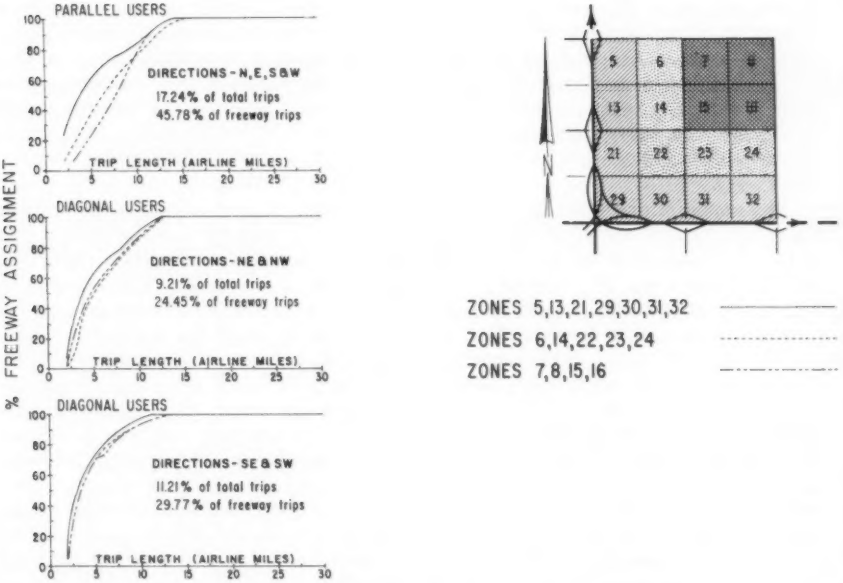

FIG. 4.—FREEWAY USERS ASSIGNMENT NORTHEAST QUADRANT 16 SQ. MILE AREA

the NE quadrant as follows: (a) Radial lines were drawn from each centroid to include 22-1/2° on either side of E and W for parallel users and 22-1/2° on either side of NE and SW for diagonal users. (b) With each centroid as a center, arcs were swung intersecting the radial lines at the half-mile points for a distance of 15 miles. It was found that trips over 15 miles had a 100% assignment to the freeways. (c) The time and actual distance were computed for the city street route and the freeway route for each zone included in a 45° sector of each mile ring. This was done for distances up to 15 miles for E and W directions (parallel) and NE and SW directions (diagonal). The other directions were assumed to be the mirror image of the first four. By taking the difference

in time and distance between the city street route and the freeway route and using the family of curves shown in Fig. 3, the percentage assignment to the freeway was determined. For each mile distance the average percent assigned was computed from the individual assignments to zones included in a 45° sector of a mile ring. These points were plotted to produce the percent freeway.

Note that only one set of curves in Fig. 4 was found necessary for parallel assignment while two were needed for diagonal assignment.

Step 8.—Using the percentage freeway assignment curves in Fig. 4, the appropriate number of freeway users for each trip length was determined and assigned to the proper access point. These one-way ADT ramp volumes are shown in Fig. 2.

Step 9.—The one-way ADT leaving the 16 sq mile area on each freeway was determined by summing the volumes shown at successive access points. Thus, in this example, the one-way ADT is 24,880. (160 + 9820 + 7660 + 7240). It should be noted that ramp volumes are shown in Fig. 2 outside of the 16 sq mile area. These are trips generated from the 16 sq mile area that would not use access points within it. If it is assumed that the area adjacent to the study is generating trips in a like manner, then the oneway ADT on the freeway would be 27,120 (24,800 + 50 + 160 + 290 + 620 + 50 + 160 + 290 + 620). The ramp volumes listed above would be increased to 260, 10,140, 8,240, and 8,480, respectively.

The assumptions used in Fig. 2 are as follows: 1. freeways spaced 4 miles apart; 2. access points to the city streets - 1 mile apart; 3. no access points to the city streets at interchanges between freeways 4. speed on freeways - 52 MPH; 5. speed on city streets - 25 MPH; 6. per cent freeway users based on Fig. 3; 7. 18,000 trip origins per square mile; 9. freeway users penalized 0.1 mile and 0.5 min at each end of freeway trips to allow for circuitous ramp travel; and 10. equal desire for travel in all directions. The results may be tabulated as: 1. freeway and ramp one-way ADT volumes as shown; 2. parallel users - 17.24% of total trips 45.78% of freeway trips; 3. diagonal users - 20.42% of total trips 54.22% of freeway trips; 4. average trip length on freeways for trips 40 miles or less = 10.74 miles; 5. sum of average approach and departure distances for freeway users = 2.43 miles; 6. average trip length of non-users = 2.99 miles; 7. average over-all trip length = 6.86 miles 5.47 miles (airline); 8. 37.66% of all trips assigned to freeways, 58.59% of total vehicle miles on the freeways, 62.34% of all trips assigned to city streets, and 41.41% of total vehicle miles on the city streets (For trips 40 miles in length or less); 9. one-way ADT on the freeways from 16 square miles for all directions = 108,480; 10. one-way ADT on a freeway from 16 sq miles for each direction = 27,120; and 11. maximum one-way ADT on the freeway system for an area over 80 miles in length and width = 85,000.

It can be seen from these results that a 4 mile spacing of freeways would provide good traffic service for the conditions given. The ramps would be operating at about maximum capacity and the 85,000 maximum one-way ADT that could develop if the system were large enough is within the practical operating range of an 8-lane freeway. An indication that a reasonably good balance of service between the freeway system and the city street has been attained is demonstrated by the results numbered 4, 6, and 8. These results should not be construed as an argument for 4-mile spacing. They are primarily intended to demonstrate certain basic relationships affecting freeway spacing. Under actual conditions of variable population densities, land use patterns, and

traffic desires, the freeway planner is more often faced with a good reason for deviating from a fixed spacing rather than adhering to it. Studies of this nature can serve an important function in the early preliminary phases of freeway system planning.

A deep appreciation by the highway planner of the complexities involved in the proper spacing of freeways in an urban area is an essential requirement for the successful prosecution of his mission to the public. Controls on freeway spacing include such physical controls as ramp location and adequate distance for traffic signing, such practical controls as availability of money for construction of the ultimate system, and such intangible controls as insuring that the planned system will enhance the growth and orderly development of the community it is designed to serve.

The highway planner must repeatedly strive for a long range view of the over-all, conflicting elements of freeway spacing or he will become hopelessly lost in a morass of technical detail that will only serve to complicate his problems. Freeways can no longer be considered on the basis of individual merit, but must be planned as a part of a system that will provide the best over-all traffic service to an area, as well as the greatest amount of community benefits.

ACKNOWLEDGMENT

The analysis of freeway access spacings, being done by the California Division of Highways, is under the direction of Gibson W. Fairman, M. ASCE.

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TRAFFIC BEHAVIOR AND FREEWAY RAMP DESIGN

By Charles Pinnell,[1] A. M. ASCE and Charles J. Keese[2]

SYNOPSIS

Highway designers must become more concerned with the relationship of design and traffic behavior in order to obtain maximum operational efficiency on freeway facilities. This paper points out the operational characteristics of freeway ramp traffic and presents requirements for correlating ramp design with traffic behavior.

INTRODUCTION

The correlation of traffic behavior and design is not a new concept. The late Thomas H. MacDonald,[3] former Commissioner of the United States Bureau of Public Roads Dept. of Commerce (USBPR) has delved into this subject. Mr. MacDonald made the following statement:

> "We have reached the point in our knowledge of the manner in which highways are used by the mass of traffic, to coordinate driver behavior under prevailing traffic conditions, and the geometric details of highway design. The degree to which the criteria so determined are accepted and intelligently applied in practice will determine the degree of safe efficiency of our future highways."

Note.—Discussion open until February 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Highways Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. HW 3, September, 1960.

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[3] "Driver Behavior—Key to Safe Highway Design," by Thomas H. MacDonald, Beecroft Memorial Lecture—Soc. of Automotive Engrs., Washington, D. C., November, 1948.

This need for correlation of traffic behavior and design is especially urgent with respect to freeway facilities.

Freeway design and operation is still a relatively new phase of highway engineering and existing freeway system have only recently experienced traffic volumes that will permit a full evaluation of their operating efficiency. The impact of the postwar growth in vehicle registrations has placed a heavy burden upon those responsible for providing adequate highway facilities and has left little time for critical review of freeway operational efficiency.

The designers of many existing freeways were faced with the difficult problem of designing a facility, whose concept was entirely new, with no operational data from which to guide the design. The efficiency and safety of operation on these facilities stands as a memorial to the ability of the highway engineers of this nation.

In the absence of operational data on freeways the designer was forced to make primary use of vehicle performance (such as size, speed, acceleration), topographical requirements, and economy as a basis for design. In most cases, this resulted in a lack of attention to the human aspect relating traffic behavior and driver requirements to design. Proper merging and diverging maneuvers can be made on most of the ramps in use today, but many were not designed so that the desired maneuvers are natural and easy and will be automatically performed by the individual drivers.

STUDY METHOD

All of the studies reported on herein were conducted through the use of motion pictures. This type of study was chosen after considering several other methods. Freeway and ramp volumes were of such magnitude and the movements so complex that other types of study produced inferior results.

Traffic operation at ramp study locations were recorded on film by use of a 16 mm motion picutre camera. The filming was done from a vantage point approximately 60 ft above the traffic stream. This position was provided by the portable tower shown in Fig. 1. The movies were made at a camera speed of 10 frames per sec which allowed accurate determination of vehicle speeds, delays, headways, and other traffic characteristics. This camera speed also permitted the reproduction of traffic operations at near actual speed.

Transverse white lines painted on the pavement provided reference points from which to determine speeds, headways and other pertinent data. The data were analyzed by use of the projector shown in Fig. 2, which was specially constructed for "still" or single-frame viewing with accurate framing and no heating or warping of the film. A frame counter and fingertip control (reverse or forward) allowed manipulation of individual frames and the determination of specific data on speeds or headways.

The motion picture studies were conducted on either a Tuesday or Wednesday to obtain average weekday conditions. Both peak and off-peak data were obtained for periods of approximately 1 hr at each study location. Ten separate ramp studies were conducted on freeways in the Texas cities of Dallas, Fort Worth, San Antonio, and Houston.

ENTRANCE RAMP DESIGN

The studies of entrance ramp operation produced much valuable information and resulted in the development of some new concepts of entrance ramp prob-

FIG. 1.—PORTABLE TOWER USED IN FILMING TRAFFIC

FIG. 2.—TIME-MOTION STUDY PROJECTOR USED IN ANALYSIS OF MOTION PICTURES

lems observed on most of the entrance ramps, were the result of a lack of consideration for traffic requirements and driver behavior in the initial design of the ramp.

Most previous ramp studies have dealt with the design of acceleration lanes. It was found in these studies, however, that the most critical area in entrance ramp design is that area traversed by the entering ramp vehicle before reaching the ramp nose. This critical area is illustrated in Fig. 3. Proper design of this area is essential to establishing the necessary correlation between design and operation.

The results of the entrance ramp studies indicated that the following factors are vital, functional elements of good entrance ramp design: Angle of entry—formed by the intersection of the entrance ramp approach and freeway lanes; Visibility relationship—between ramp traffic and freeway traffic; Delineation—ramp nose and acceleration lane.

Angle of Entry.—The initial ramp studies were conducted on the Gulf Freeway in Houston, Texas, at a location where the ramps had an angle of entry of approximately 14° degrees, as shown in Fig. 4. It was found that this high angle of entry aimed the entering drivers into a direct-type entry path onto the main freeway lanes. A typical maneuver of this type is shown in Fig. 5. A majority of drivers prefer to follow the easy, direct path along which they are aimed rather than make the proper but more difficult maneuver of turning along the acceleration lane.

Studies of vehicle entry paths indicated the ramp usage pictured in Fig. 6. As can be seen, a relatively small per cent (8.65) of the drivers utilized the acceleration lane. The predominant direct-type of entry results in the creation of a high relative speed between entering ramp traffic and freeway traffic. This high speed differential creates undesirable friction in the freeway traffic stream and produces a large number of weaves in the ramp area resulting in inefficient, hazardous operation.

Modifications were made in the ramp design (Fig. 7) to allow the extension of the acceleration lane to approximately 1,000 ft in length. This was done in order to determine if an increase in the length of the acceleration lane would improve ramp operation. The usage of this ramp is shown in Fig. 8. The direct, high relative speed type of entry was reduced by 21.17% but was still being made by 70.18% of the entering drivers.

Thus, the studies indicated that the primary influence on the path followed by an entering driver is directly related to the angle of ramp approach. If desired usage of the acceleration lane is to be obtained, it is necessary to align the driver along the acceleration lane. The driver can then follow a natural and easy path into the freeway utilizing the acceleration lane as a speed adjustment area and performing a simple lane-change maneuver into the freeway. This can be accomplished by providing a flat-angle ramp approach as indicated in Fig. 9.

Studies were also made on entrance ramps having a more desirable design and on one ramp which had been modified with paint striping to align drivers along the acceleration lane. These studies indicated an increased use of the acceleration lane and smoother merging of the ramp and freeway traffic. A typical entry of this type is shown in Fig. 10.

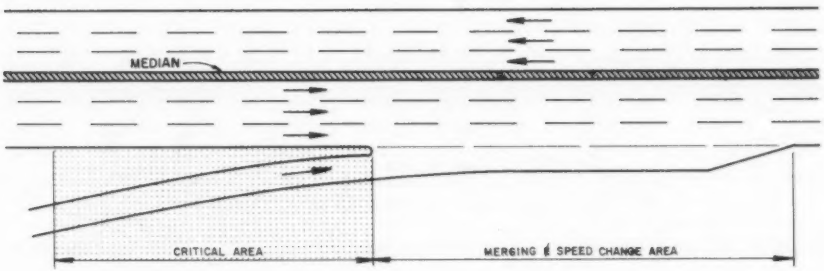

FIG. 3.—CRITICAL AREA RAMP DESIGN

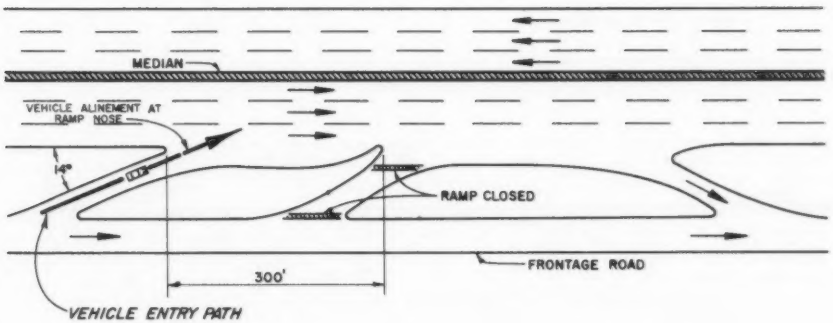

FIG. 4.—CULLEN ON RAMP GULF FREEWAY-HOUSTON

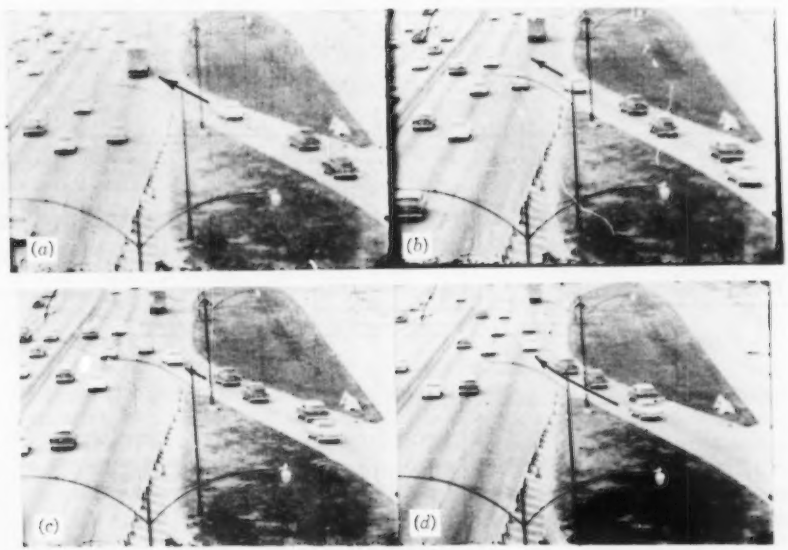

FIG. 5

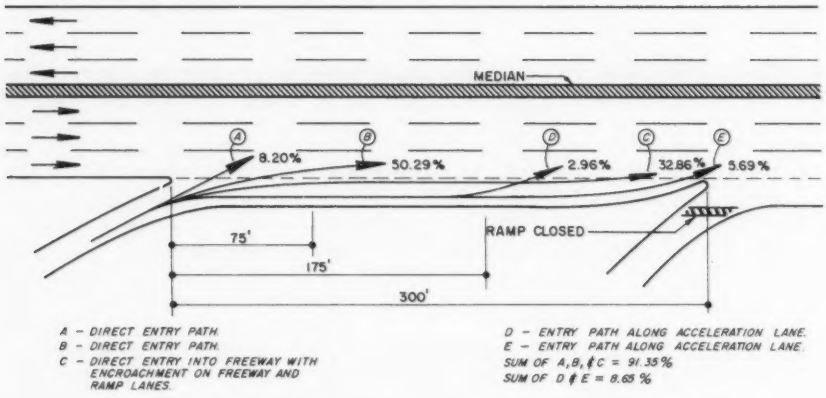

FIG. 6.—PATHS OF ENTRY SHORT ACCELERATION LANE

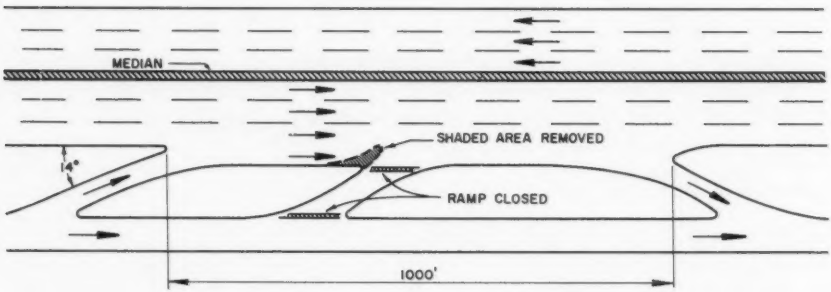

FIG. 7.—CULLEN ON-RAMP MODIFIED NOSE

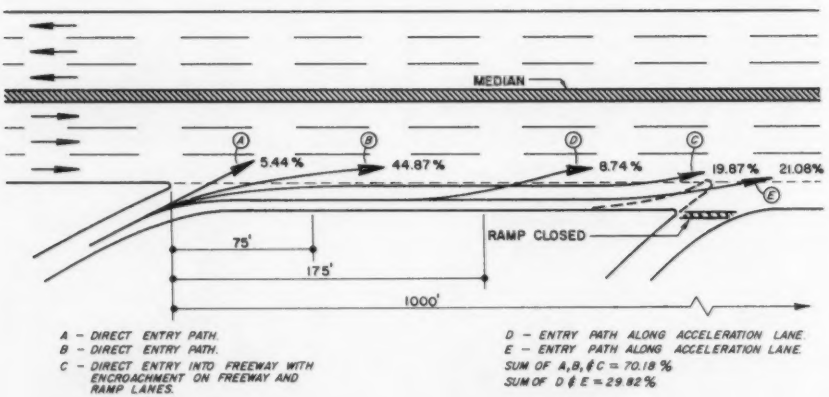

FIG. 8.—PATHS OF ENTRY LONG ACCELERATION LANE

Visibility Relationship.— The visibility of main freeway lanes and freeway traffic provided to the entering ramp traffic is another integral part of good freeway ramp design. Studies of traffic behavior on entrance ramp approaches where good visibility was available, indicate that the driver begins his evaluation of the freeway traffic stream and makes his decision regarding freeway entry as far as 200 ft before reaching the ramp nose.

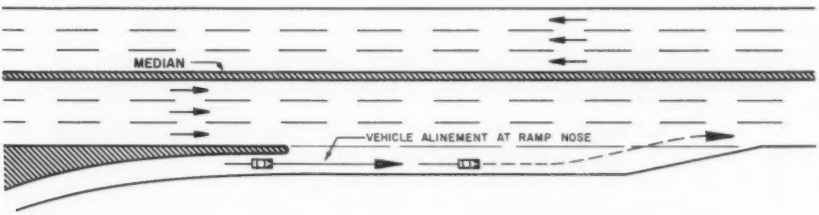

FIG. 9.—FLAT ANGLE APPROACH

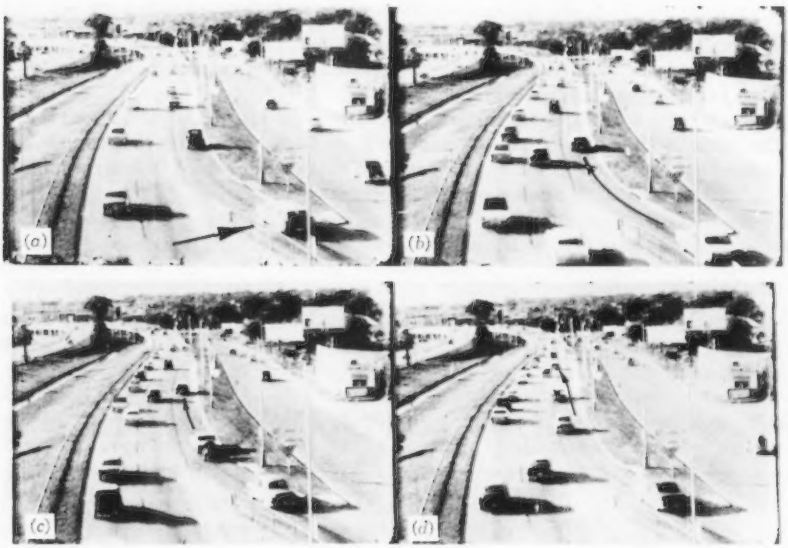

FIG. 10

This causes some changes in the concept of the manner in which ramp traffic enters a freeway. Instead of arriving at the nose at a slow speed and then accelerating along the acceleration lane, the most desirable entry maneuver is made by the driver who approaches the ramp nose at a fairly high rate of speed. This driver selects a gap in the freeway traffic stream before reaching the ramp nose, adjusts his speed if necessary on the acceleration lane and performs a smooth "lane change" maneuver into the freeway.

If the driver is unable to observe the freeway in the proper perspective before reaching the ramp nose, then it is usually necessary for him to stop at the ramp nose and be delayed while selecting a freeway gap. Fig. 11 shows the behavior of an entering driver that has stopped at the ramp nose and is looking back over his shoulder to judge a freeway gap. A high relative speed exists between his vehicle and the main freeway traffic and his maneuver into the freeway traffic stream is difficult and hazardous. Fig. 12 shows the behavior of another driver, entering the freeway at the same location, who has selected a gap before reaching the ramp nose and moves into the freeway with a minimum of effort and with little or no effect on freeway traffic.

Care must be exercised in the location of ramps to prevent any obstructions to view. A typical problem that can develop is shown by the interchange design in Fig. 13. In this case, the freeway is over the major street and the freeway entrance ramp is introduced just past the overhead structure. As indicated by the grade profiles (Fig. 13) and the photograph (Fig. 14(a)), a very bad sight restriction exists. It is impossible for the drivers on this ramp to evaluate freeway traffic before reaching the ramp nose. As a result, a majority of the drivers must stop at the ramp nose before entering the freeway.

Freeway entrance ramp grades should be treated as shown in Fig. 15. The entrance ramp grade (either ascending or decending) should be made to match that of the freeway for a minimum of 250 ft in advance of the nose. This provides the ramp traffic with adequate visibility, as illustrated in Fig. 14, and permits the entering ramp traffic to begin evaluation and selection of freeway gaps before reaching the ramp nose.

Delineation.—A third factor of importance in entrance ramp operation is delineation of the ramp nose and acceleration lane. While the increased use of freeways has resulted in a larger number of drivers educated to their use, there are still many drivers who do not fully understand the proper method of using various freeway elements. Also, there are so many different types of entrance ramps (without acceleration lanes, with short acceleration lanes, long acceleration lanes, and with additional freeway lane at ramp) that the driver often does not know what to expect.

The ramps studied were not well delineated and, in many cases, the actions of the drivers indicated they did not know the acceleration lane was exclusively for their use.

Good delineation—in the form of color contrast, paint striping or other methods—should be provided to enable the drivers to adequately distinguish between the main freeway lanes and the portions of the entrance ramps specifically designed for their use.

ENTRANCE RAMP ACCIDENTS

Studies of accidents on freeway facilities in Texas[4] with special emphasis on correlating accidents with design elements have indicated that entrance ramps often experience a high accident frequency. A typical accident pattern on an entrance ramp is shown in Fig. 16. The predominant accident type is the rear-end collision.

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4 "Freeway Traffic Accident Analysis," by B. F. K. Mullins, and C. J. Keese, Committee Report, Highway Research Bd., January, 1959.

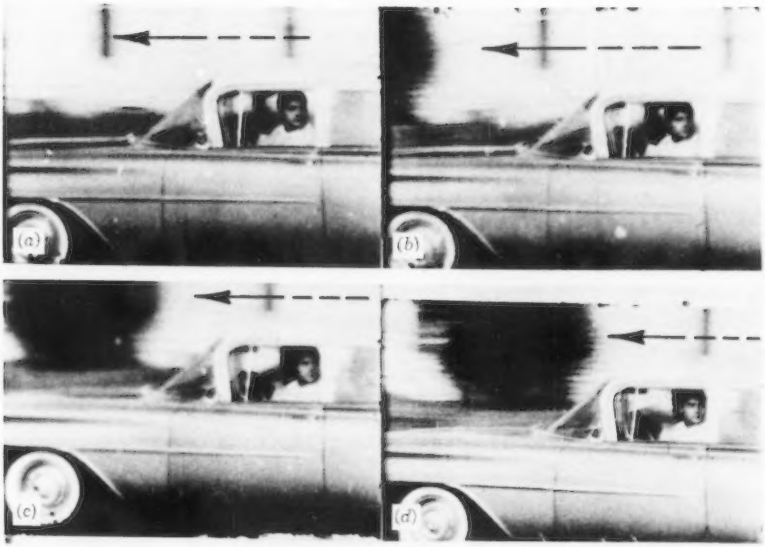


FIG. 11

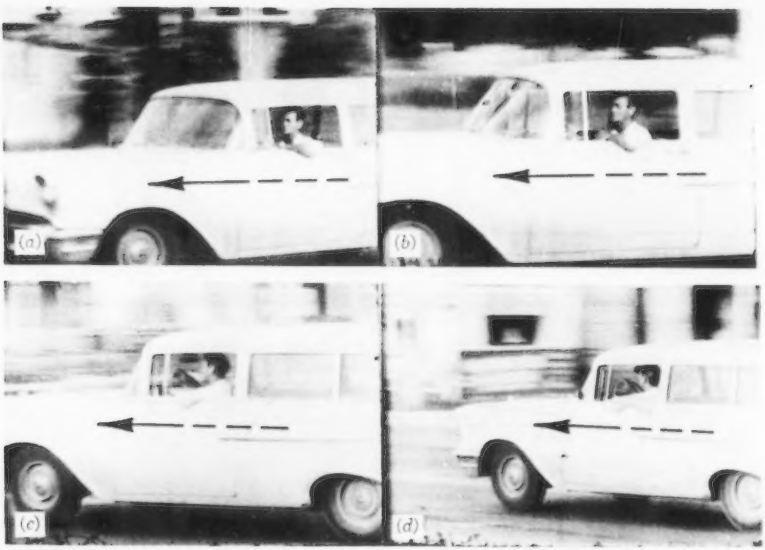


FIG. 12

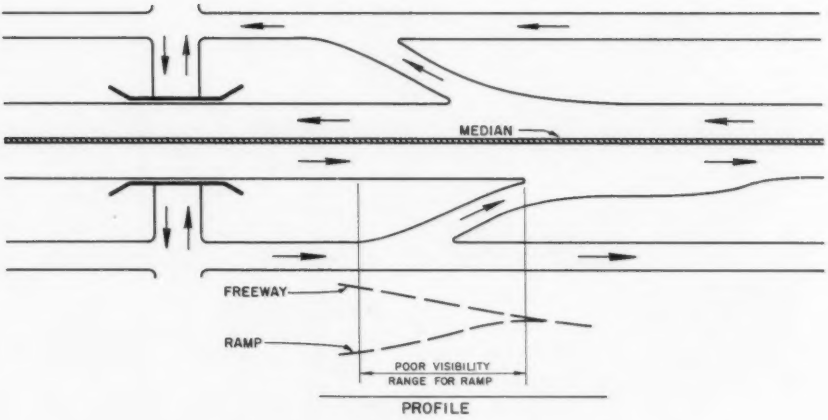


FIG. 13.—DIAMOND INTERCHANGE

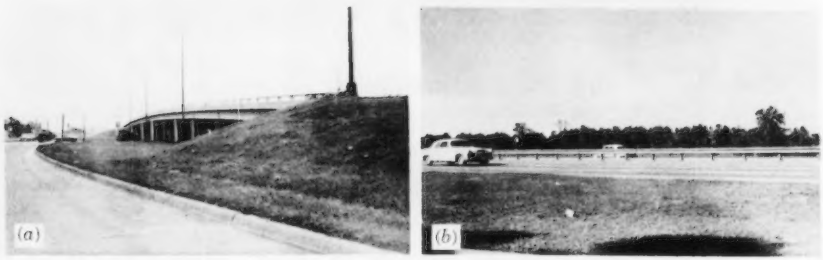


FIG. 14

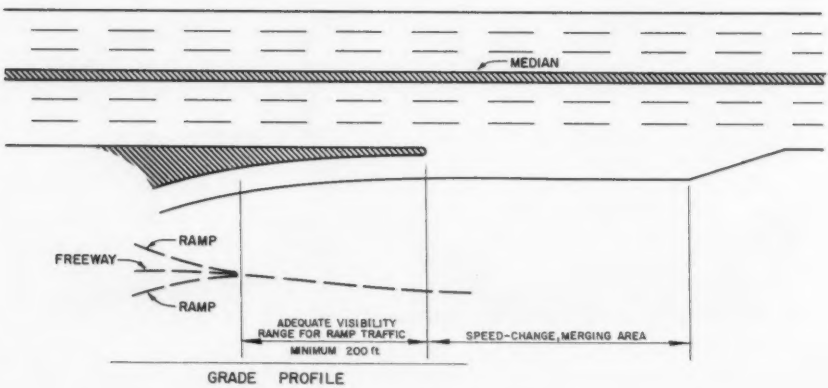


FIG. 15.—RAMP GRADE

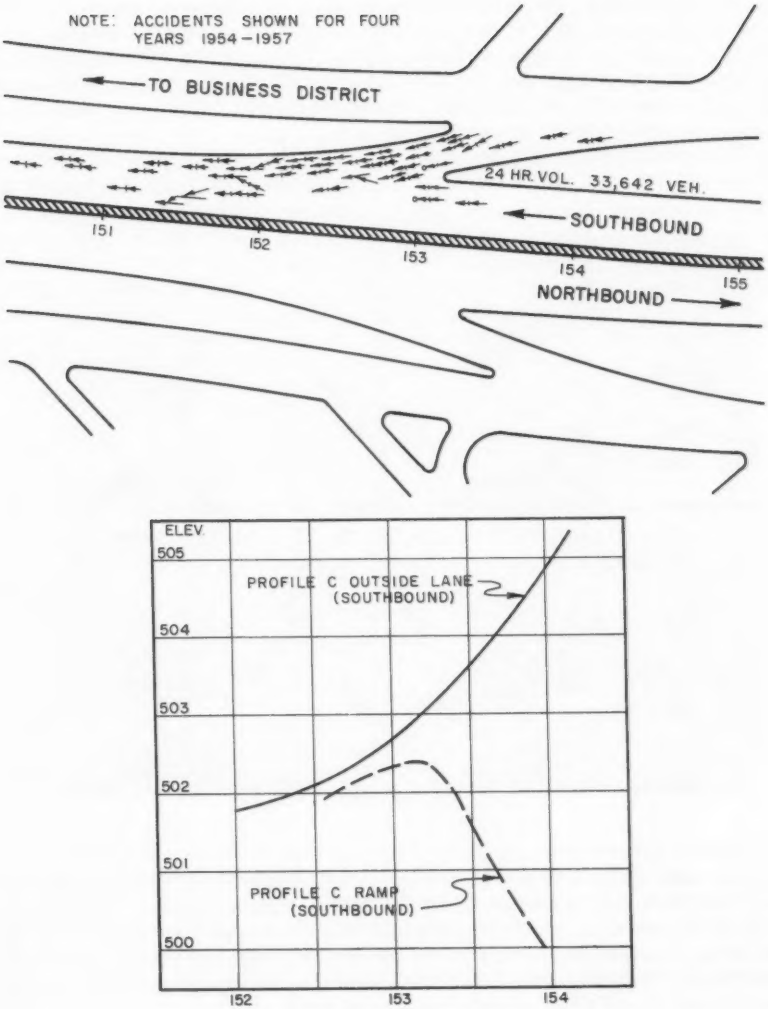


FIG. 16.—CENTRAL EXPRESSWAY-DALLAS, TEXAS
ON-RAMP AT CALVARY

During the various study periods, five ramp accidents occurred and were recorded on film. A typical entrance ramp accident is illustrated in Fig. 17. Here, as in the case of most entrance ramp accidents, the accident resulted from a "false-start" by the leading vehicle and a collision by the trailing vehicle. In this case, the driver of the trailing vehicle is put in the unfortunate position of needing to look two directions at the same time. He must accept a gap in the freeway stream and also keep an eye on the vehicle ahead. The trailing driver often assumes that the lead vehicle is going into the freeway and looks back while his vehicle is moving forward. If the lead vehicle stops, a rear-end collision often results.

The basic cause of this accident, however, is related to the fact that many of the vehicles stop at the ramp nose. The cause of this stop is related to the

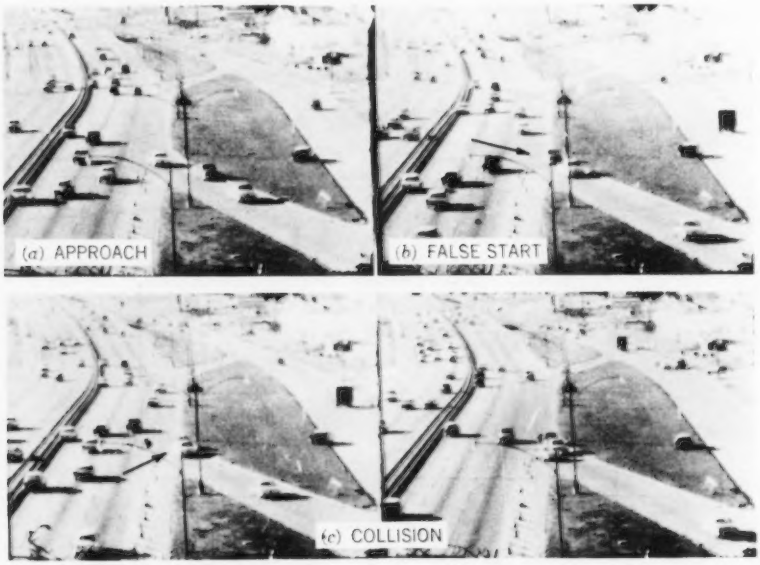


FIG. 17

lack of design consideration for the previously mentioned factors of approach angle, visibility and delineation. If the drivers were properly aligned along the acceleration lane, provided adequate visibility, and were sure of the purpose of the acceleration lane, a majority of them would move into the freeway in a continuous movement. Even if a gap were not available in the freeway stream there would be no need for an abrupt stop.

Thus, the inclusion of the proper design elements should serve not only to produce a smoother merge of the ramp and freeway traffic but should also aid in reducing the accident problem that exists on many entrance ramps.

RAMP APPROACH WIDTH

Studies were conducted on entrance ramps with wide approaches such as the one shown in Fig. 18. The approach width on this ramp is wide enough to allow

two lanes of traffic to approach the ramp nose. Though the ramp is not marked for two lane operation, traffic frequently approaches the ramp nose in this manner.

Several ramps with wide approaches were studied, and in all cases unsatisfactory operation was indicated. Since only one vehicle at a time can utilize the acceleration lane, the two lanes of ramp traffic must merge into single file or else the vehicles in the left lane of the ramp are forced to make a direct entry into the freeway without the benefit of the acceleration lane. A typical illustration of this operation is shown in Fig. 19. In this case, a car in the inside lane is waiting to make a direct entry and is blocking the visibility of the other ramp traffic. The traffic behind this car attempts to change lanes in order to reduce their delay and a hazardous situation develops.

These studies indicated that much better ramp operation is obtained on ramps designed to allow only one lane of traffic. The approach section should be designed with a mountable curb and shoulder on the right to allow disabled vehicles to leave the traffic stream and thus eliminate the need for providing ramp width for emergency storage.

Where ramp demand exceeds the capacity of a single lane ramp, special considerations will be required.

DESIRABLE ENTRANCE RAMP DESIGN

Fig. 20 shows an entrance ramp design incorporating all of the features previously discussed. This design should provide the necessary correlation between traffic behavior and design to obtain the maximum efficiency in entrance ramp operation.

The desirable length of acceleration or "speed adjustment" lanes is a point of question and additional research must be done in this area. The data available at present indicates that the length can be reduced, and the design indicated in Fig. 20 is a trend in this direction.

EXIT RAMPS

Studies of exit ramp operation also indicate a need for special consideration of traffic behavior in the design of off-ramps. The studies indicate that safe, efficient exit ramp operation is related to (a) provision of a natural exit path with adequate deceleration distance provided past the ramp nose, (b) provision of adequate sight distance on the exit ramp to allow evaluation of the conditions at the ramp terminal, and (c) delineation of exit ramp nose and deceleration area.

Natural Exit Path.—The two most prevalent types of exit ramps are shown in Fig. 21. The basic difference in the two ramps is in the provision for vehicle deceleration. One allows for deceleration on a parallel lane adjacent to the main freeway lanes and the other type provides for deceleration off of the main freeway lanes.

Fig. 22 shows the exit paths of vehicles using an exit ramp on the Gulf Freeway. At this location a parallel lane is provided for deceleration but receives

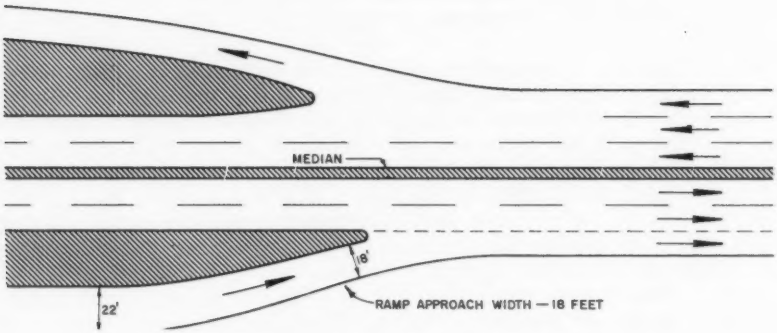


FIG. 18.—ENTRANCE RAMP WITH WIDE APPROACH

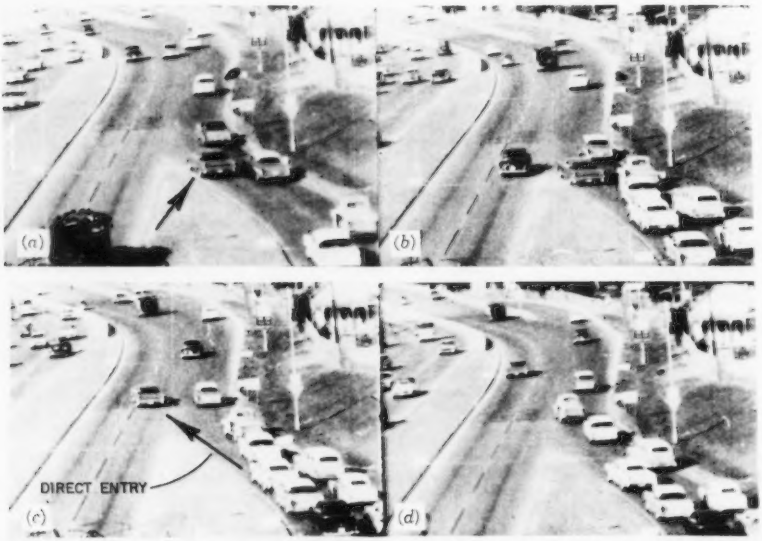


FIG. 19

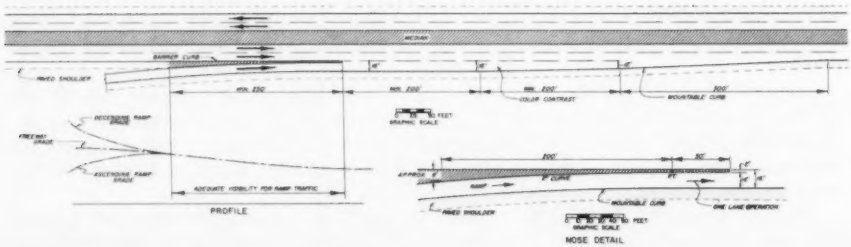
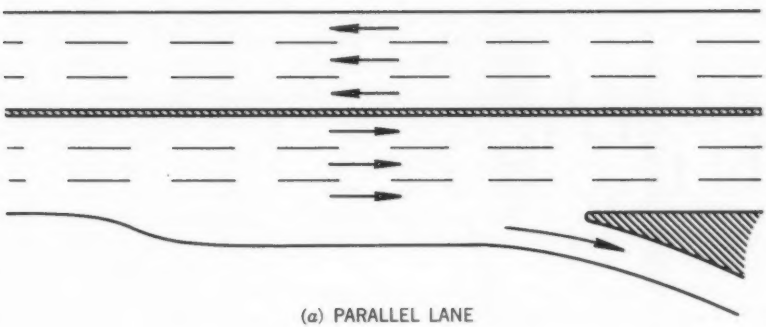
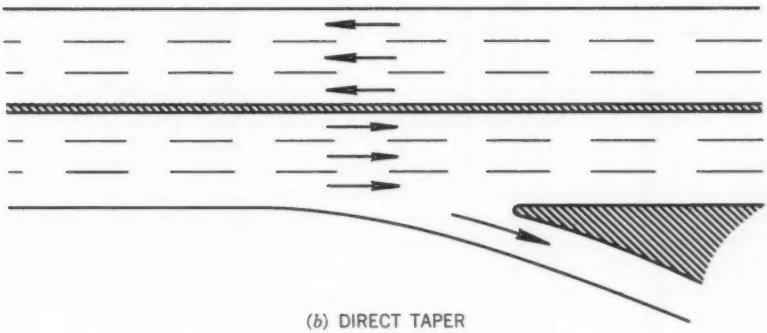


FIG. 20.—DESIRABLE ENTRANCE RAMP DESIGN



(*a*) PARALLEL LANE



(*b*) DIRECT TAPER

FIG. 21.—EXIT RAMP TYPES

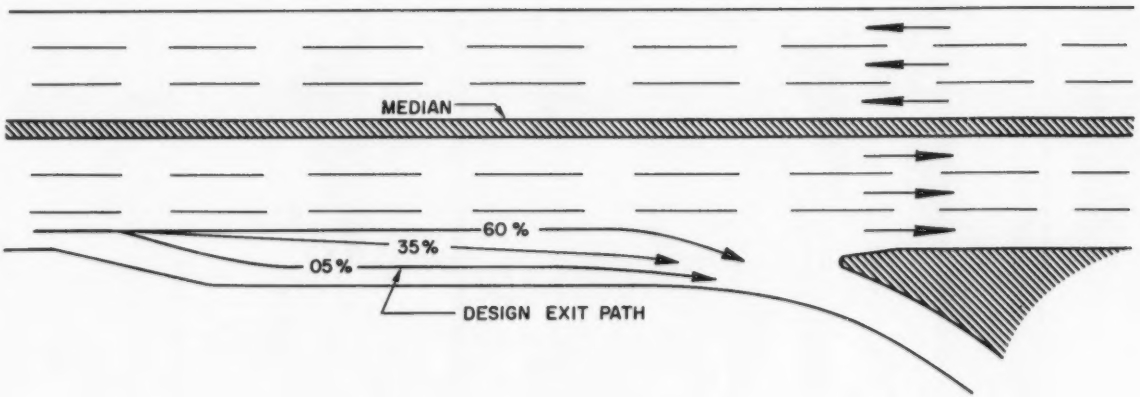


FIG. 22.—EXIT PATHS

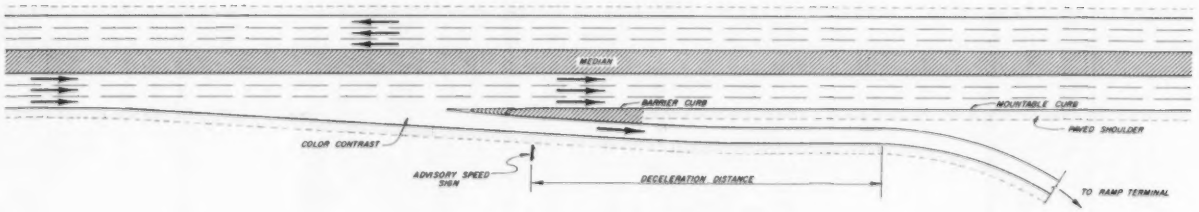


FIG. 23.—DESIRABLE EXIT RAMP DESIGN

little use as indicated by the vehicle paths. This lack of usage is related to the exit ramp driver's desire to follow a natural and easy path. Use of parallel deceleration lane requires a reverse curve movement, which represents additional maneuvering to the driver.

Thus, if design of exit ramps is to be fitted to actual traffic behavior, a natural, direct-type off-ramp appears to be necessary. Other studies[5] have also indicated the desirability of the direct-type exit due to the minimum effect on the freeway traffic stream. If the ramp is designed to accommodate traffic exiting at freeway design speed, then the exit maneuver is accomplished with a very low relative speed and little or no effect on the main freeway traffic.

Provision of Adequate Sight and Deceleration Distance.—If the design allows the off-ramp traffic to exit from the freeway at design speed, as previously described, careful consideration must be given to the design of that portion of the exit ramp that lies past the ramp nose. A sufficient deceleration distance must be provided and adequate sight distance should be available to allow the driver to judge conditions and adjust his vehicle to a proper speed for traversing the terminal area.

Due to variations in interchange designs and ramp connections there are many different types of ramp terminals confronting exit drivers. Each type of terminal may require different operation of the vehicle. The exit ramp should be designed so that the driver will be able to safely maneuver through the terminal area regardless of its design.

Delineation of Ramp Nose and Exit Area.—Care must also be exercised to assure that the driver can adequately distinguish the exit ramp and be able to make the proper maneuver. The driver can be materially aided in this respect by providing good contrast and delineation of the exit ramp. This can be accomplished by paint, paint striping and color differences between main freeway lanes and exit ramp area.

DESIRABLE EXIT RAMP DESIGN

Fig. 23 shows an exit ramp design that incorporates the features previously described. This design correlates the needs and desires of traffic with design features to obtain maximum safety and efficiency of operation. Since the design will vary slightly with such features as design speed, terminal conditions, and so on, no specific figures or dimensions have been indicated. The suggested design shown, however, could be adapted to specific conditions as required.

RAMP LOCATION

Many operational problems on freeways have been attributed to the frequency of ramps, when in fact the problems are more often related to inadequate ramp design. Ramps are often placed in locations in which distance limitations or profile conditions make adequate ramp design impossible. Thus, one of the

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5 "A Comparison of Vehicle Operating Characteristics Between Parallel Lane and Direct Taper Types of Freeway Off-Ramps," by Robert D. Conklin, Traffic Engineering, December, 1959.

major considerations given to selection of ramp locations should be the requirements for providing an adequate ramp design.

SUMMARY

Relating the design of freeway ramps to traffic behavior as indicated by the requirements and desires of the drivers is a necessity if maximum efficiency and safety in ramp and freeway operation is to be obtained.

Entrance ramp design should provide the following:

1. A ramp approach that aligns the driver along an easy and natural entry path into the freeway.
2. Adequate sight distance to allow the entrance ramp driver to judge and accept a freeway gap with a minimum of effort.
3. A clearly marked and delineated entrance ramp that will eliminate any confusion in distinguishing between the entrance ramp elements and the main freeway lanes.

Exit ramp design should provide the following:

1. A natural and easy exit path that will require a minimum of driver maneuvering.
2. Adequate deceleration and sight distance past the ramp nose to allow the driver to safely maneuver through the ramp terminal area.
3. Good delineation at the exit ramp area to eliminate any confusion in distinguishing between main freeway lanes and exit ramp area.

ACKNOWLEDGEMENTS

This paper is based upon the results of research studies conducted by the Texas Transportation Institute in cooperation with the Texas Highway Department. Grateful acknowledgement is made to representatives of the Texas Highway Department and the cities of Houston, Dallas, Fort Worth, and San Antonio for their cooperation, valuable advice, and assistance.

Gratitude is also expressed to Fred H. Edwards, graduate student in civil engineering at Texas A and M College, who made significant contributions to the study.

ADDITIONAL READING REFERENCES

1. "Operational Characteristics of High Volume On-Ramps," by Eugene J. Lessieu, Proceedings, Inst. of Traffic Engrs., September, 1957.
2. "Traffic Behavior as Related to Several Highway Design Features," Highway Research Bd., Bulletin 170, 1958.
3. "A Study of Freeway Traffic Operation," by Charles J. Keese, Charles Pinnell, and William R. McCasland, Highway Research Bd., January, 1959.
4. "Spacing of Interchanges on Freeways in Urban Areas," by Jack E. Leisch, Proceedings, ASCE, Vol. 85, No. HW4, December, 1959.
5. "Operational Aspects of Controlled Access Facilities," by Joseph E. Havenner, Proceedings, ASCE, Vol. 85, No. HW2, April, 1959.

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HIGHWAY DIVISION
Proceedings of the American Society of Civil Engineers

DISCUSSION

Note.—This paper is a part of the copyrighted Journal of the Highways Division, Proceedings of the American Society of Civil Engineers, Vol. 86, HW 3, September, 1960.

PASSENGER DATA FOR URBAN TRANSPORTATION PLANNING[a]

Discussion by Henry A. Barnes, John Rannells, Charles M. Noble, Hawley S. Simpson, Verden A. Rittgers, Seymour Kashin

HENRY A. BARNES.[1]—The problem pointed out by the author is one which is most pertinent in the city of Baltimore. It is evident that railroads hold little hope as a solution to mass transit woes since this inflexible medium of transportation can only be justified for carrying passengers over relatively long distances between high concentrations of residential and industrial land use. The existing rail network is not oriented to serve such concentrations and the distances involved in travel in the Baltimore Metropolitan Area would militate strongly against any outlay for fixed-wheel transit.

A circle drawn with the Central Business District as the center and a radius of about 6 miles would encompass well over 90% of the people working in the city. It would seem that buses also face the problem of land use concentration and that these facilities have the advantage over rail in that their flexibility permits them to serve essentially both express and local service at the same time.

The author points up the growth of reversed travel and peripheral travel. These changing trends are strictly brought out in the Baltimore Area from a recent study, conducted in Baltimore, based on the Gravity Model Analysis of O and D. The futility of trying to provide for metropolitan growth by continuing to consider the area as central-business-district dominated is evident. Future demands for reverse travel and circumferential movements will distribute trips over so wide an area that this travel must, of necessity, be principally handled by private automobiles with mass transit primarily serving the downtown area.

JOHN RANNELLS,[2] M. ASCE.—Mr. Cherniak's analysis of trans-Hudson passenger data, 1948-58, is excellent in its clarification of differences among various segments of travel differences which, if not taken fully into account, could lead to false conclusions regarding the relative importance of competing modes of travel. The reader must keep in mind, however, that this paper treats of data in just one specialized portion of New York's transportation planning problem and that the author's conclusions do not apply to other parts.

These conclusions, indeed, derive from the very existence of the Port Authority's magnificent properties, which are handsomely supported (and in turn support other properties) by levies on the highway user—levies that the highway user has been quite willing to pay, it must be admitted. The question

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[a] December, 1959, by Nathan Cherniack.
[1] Commissioner, Dept. of Transit and Traffic, Baltimore, Md.
[2] Asst. Dir., Urban Traffic Transp. Bd., Philadelphia, Pa.

whether, in "the grand accounting," movement across the Hudson might be better served by something different—for example, a combined rail-highway program—is avoided in this paper. But this question is, nevertheless, the spectre that looms behind the Authority's trans-Hudson properties.

The author is quite correct in stating that the problems of urban transportation are too complex to be solved by setting up an over-all operating organization. But full integration, in this sense, is not required in order to achieve better utilization of all means of passenger transport at lower total cost to the public, both as consumers and as taxpayers.

That financially strong segments of the urban transport system should be kept strong at the expense of weaker segments is a proposition that will not bear scrutiny, for the general public pays the total bill. If New York City is to continue viable, means must be found for more balanced utilization of mass transport facilities by selective development of the more useful and potentially economical segments of the passenger—rail network. This need not act to the detriment of the Authority's trans-Hudson properties, although it would diminish their present dominance.

In short, this paper is excellent in its treatment of the trans-Hudson passenger data and in its discussion of the broad range of materials needed for urban transportation planning, but its conclusions are based on too-special a selection of experience.

CHARLES M. NOBLE,[3] F. ASCE.—The paper indicates trends in urban metropolitan area transportation needs that must be heeded if incorrect decisions in urban and transportation planning are to be avoided. It makes clear that transportation planning must envision things to come rather than accept things as they are, since a cursory look at present travel may be misleading in forecasting the future. The trends for the future must be dug out of the mass of statistics emerging from the electronic computer and must be correctly interpreted.

The responsibilities laid on the transportation engineer are heavy and solutions cannot be found by superficial analysis. The problem today is far more complex and subtle than in 1920, when the automotive era really began. Then the nation's economy revolved around rail and water transportation and the mold and pattern of urban living, commerce, and industry was stabilized around fixed rails and water channels. The free-wheel automotive vehicle, together with aviation, have revolutionized transportation trends and demands. The public now has free choice and fixity has given way to flexibility. This has transformed the economy of America and has affected land useage, public habits, commerce, and industry. There are no easy cliches and pat solutions to the staggering transportation problems facing the nation. The advancing gross national product and the post war population explosion bring these problems into critical focus.

Fluidity and freedom of choice are upsetting past ideas concerning land uses. Yet, the transportation engineer must base his planning on the land uses of the future, in spite of the fact that trends are not too well understood at this date.

Provision of transportation media will affect an area and will determine land uses whether the planner likes it or not. But it is in the public interest and will be of the greatest benefit to the national economy if transportation

[3] Cons. Engr., Princeton, N. J.

media are built in locations that will develop the land for its most effective uses, not only from an economic standpoint but also to satisfy aesthetic values and to provide the population with better living standards.

Regional planning and zoning are a must in this day of population explosion and economic expansion. Without it, there can be nothing but chaos in metropolitan areas. The many local communities that make up such areas cannot be expected to see the overall picture and plan effectively in relation to the entire region. The problem will be to achieve regional planning and zoning so that the overall plan will be effectuated.

Transportation facilities are too costly today to be provided on a hit or miss basis. Only sound planning will assure that capital investment will be wisely spent and that the transportation media will be a success. The nation can no longer afford costly failures. Future land uses and future employment centers must be predicted accurately and transportation facilities planning tailored to that pattern utilizing each form that is suitable and practical, ever being mindful that the preferences and convenience of the public will be decisive.

Mr. Cherniack's important paper shows the trend that heavy radial movements of people to the central business district in the New York-New Jersey-Connecticut metropolitan area are declining, and that "circumferential" movement is developing.

Studies by the Pennsylvania Railroad and others show that rail commuter services cannot survive on a profitable basis on the "twenty hour" week, where the rails handle dense volume for about 2 hr in the morning and again in the evening for 5 days per week. Personnel, equipment and plant are idle the remaining hours.

It seems axiomatic that fixed rail facilities, including rapid transit, would be even less profitable if it were required to serve the periphery. Thus, it appears that fixed rail facilities can only be profitable under dense traffic spread over day and week, and certainly would not be economically feasible even with a subsidy to serve the just developing peripheral passenger movement in less densely populated areas over considerable distances.

However, the opinion expressed by the writer is not shared by all as evidenced by the following quotations from Charles J. Lietwiler, a transit enthusiast, who was asked by the writer to comment on Mr. Cherniack's paper.

> "The basic principles of transportation would tell you that the area served by the rail lines would become densely settled first. Since the commuter service developed along lines that were originally constructed for freight and through passenger service, and since the commuter operations have generally been unprofitable since the 1920's, little, if any, extensions of service have been made. With the development of the automobile and the increase in population, the unsettled areas not served by the rail system became relatively more attractive to developers and householders than the already thickly settled areas served by rail lines. Naturally, these newer sections are going to have greater increases than the rail-served areas, and increases in bus and automobile travel. This is hardly a valid basis for the negative attitude toward rail transport that appears throughout this paper."

> "Building highways and inducing people to drive automobiles is one thing. To base future construction on the induced highway travel without regard to the effects on the economic center toward which this travel is directed is to consider only a part of the problem, and out of context at that."

"Must we continue to build these low-capacity (in terms of the people) high-cost facilities simply because they are in use twenty-four hours a day? Has continuous use become such a virtue that it takes precedence over total cost to the community or total unit cost to the individual in terms of service produced?

"A railroad track on which both freight and passenger service are being operated is certainly in use more than '2 or 3 morning hours' and '2 or 3 afternoon hours.' I believe that the utilization of existing tracks will increase over the years rather than decrease."

"There is still some passenger service other than just commuter trains."

"The attitude of the author, as reflected in this paper, is this: We had nothing to do with the present situation, and there is nothing we can do about it now (or want to do about it). The trend that now exists is the 'right' one, so we should accommodate that trend. This attitude fails to recognize the basic transport problem, which is the movement of people, not the movement of automobiles, buses, or trains."

"If a decision is made that the overuse of automobiles by commuters is endangering the community as a whole, then the necessary steps must be taken to see that automobile commuters are transferred onto other modes of transportation (by choice, if possible). There is no basic difference between this decision and a decision to ban the dumping of garbage in city streets, or a decision to prohibit the keeping of livestock within the city limits."

It seems clear, however, and it is believed that it will be accepted by the unbiased, that the entire transportation needs of an area must be developed and each form applied where it satisfies the need and is practical to the end that the most economical plan for the entire area is selected. Such a plan cannot be considered practical unless it meets the desires and preferences of the public on a free choice basis. It goes without saying that the economic effect on the communities involved must be placed in the formula. Loss of tax rateables due to right-of-way requirements, deterioration of tax values caused by noise and other adverse factors must be given full weight.

However, before the nation rushes into a vast program of rail rapid transit, extensive basic research should be conducted to develop a rail vehicle that is acceptable to the public, noiseless, dustless, comfortable, attractive, fast, economical. There has been no basic development in America of the rail vehicle for fifty years. By contrast, the automobile industry has expended vast sums for research and produces a new model each year. They concentrate on selling the customer the vehicle. The rail industry does not sell a vehicle, they purvey a service. Often this service is not acceptable to the public.

The public is the key and the transportation system must be geared to its wishes and preferences.

The author is to be congratulated for the presentation of this enlightening paper, and it is hoped that the paper will stimulate publishing of similar studies made in different parts of the country. In this way, the new trends will become clearer and realistic transportation planning will be a closer reality.

HAWLEY S. SIMPSON,[4] F. ASCE.—Mr. Cherniack's paper concludes with a very pessimistic view regarding the future of rail rapid transit when he says:

[4] Simpson and Curtin, Transp. Engrs., Philadelphia, Pa.

"Today, capital investments in new systems of rail mass transit facilities do not appear to be justified."

The quoted statement is unsound because it can be demonstrated that the investment in rail rapid transit is less than for either automobile or bus, under comparable conditions of speed, capacity, and central area delivery.

For example, the Congress Street Expressway in Chicago (four lanes in each direction) cost $10,000,000 per mile. The added cost of rapid transit right-of-way (space for two tracks in each direction) was less than $500,000 per mile; the total rapid transit cost (one track in each direction) was less than $4,000,000 per mile.

In November, 1959, the Congress Street Rapid Transit was carrying about 10,400 passengers at the peak-hour inbound; and the same number outbound. Automobiles were moving 7,700 people inbound, and 6,500 outbound in the same time.

Thus transit, operating at about 25% of its capacity of 40,000 persons per hr, is carrying about 50% more persons than the paralleling vehicle expressway operating in excess of its "practical capacity."

Even on a 24-hr basis, Congress Street Rapid Transit, with one track in each direction is carrying half as many passengers as the Expressway in four lanes.

Furthermore, rapid transit patrons create no demand for central area car-storage facilities.

In Toronto $60,000,000 has been spent since World War II, on a modern rapid transit subway on Yonge Street; an additional $200,000,000 is being invested in a subway on Bloor Street with a spur paralleling Yonge Street for downtown delivery.

Increased ratables along Yonge Street are producing $5,500,000 annually in additional tax revenues. Property values there have risen twice as fast as in the rest of the city.

In both Chicago and Toronto, the rapid transit lines have been built to meet modern-day transportation requirements. Mr. Cherniack's comparison between highways and rail transit for trans-Hudson trips in based on a comparison of highway facilities which have been vastly improved and expanded in recent years, whereas the basic facilities for rail travel have not been measurably improved in the past half century. The last trans-river crossing of rail facilities was completed in 1909: the Hudson Tubes.

The conclusions drawn by Mr. Cherniack's article are based on an analysis of a situation quite unique in transportation annals. Until the opening of the Holland Tunnel, in 1927, the entire North Jersey section of the New York metropolitan area was physically isolated insofar as vehicle travel was concerned. Vehicles crossing the Hudson relied solely on ferries. The need for bridge and tunnel crossings to provide adequately for the movement of people and goods across the Hudson River in interstate commerce was obvious. Naturally, these facilities have made commuting by automobile or bus relatively more attractive than in the pre-Holland Tunnel era. This, combined with the fact that nothing of substance has been done to better commuter rail travel across the Hudson, has profoundly effected the trans-Hudson traffic pattern. Mr. Cherniack's paper must be viewed in the light of these rather exceptional circumstances.

Except for the traffic on the Pennsylvania Railroad between New Jersey and Manhattan, all rail passengers between New Jersey and Manhattan must make at least one transfer, either to the Hudson Tubes or to a cross-river ferry.

That infirmity is not attached to either automobile or bus travel, as the result of the construction of many costly facilities for uninterrupted cross-river travel by tunnel or bridge.

Thus, automobiles can deliver directly to a point near any Manhattan destination; buses can deliver to the 34th-42nd Street area, but a transfer is required to most central destinations because the Port Authority Terminal is near the western edge of Manhattan, three-quarters of a mile from Fifth Avenue.

The importance of ready delivery to destination areas is clearly exemplified on lines 6, 7, and 8 of Table 4 of the paper.

South of Houston Street, where delivery by rail is better than in any other area, 72.8% of travel in rush hours is by rail; from Houston to 34th Street where delivery is not so good, 42.5% of rush hour travel is by rail; between 34th Street and 59th Street where a second transfer is required if rail facilities are used, only 27.1% of rush-hour travel is by rail. On the other side of the coin it will be noted that the only area in which more than half (in this case 57.5%) of rush-hour travel is by buses is above 34th Street, the area within which the Port Authority Terminal is located.

The importance of delivery to the destination area is a matter that is often overlooked, but it is a factor which has a very great influence on the distribution of person-trips by modes of travel.

Turning to the east side of Manhattan and examining travel to and from areas to the east, it has been reported that rail travel between 1940 and 1955 from Westchester and Fairfield Counties has increased from 42,000 in the commuter period (7 a.m to 10 a.m.) in 1940, to 60,000 in 1955. The increase in absolute numbers of passengers is almost identical with that by highway. Thus, despite the very considerable improvement in highways, with practically no change in the type of rail transportation, the growth in travel into the city in commuter hours has been divided almost equally.

From Long Island to Manhattan, although substantial improvements in highway facilities between Long Island and Manhattan have been constructed, rail travel has expanded from 48,000 in the 7 a.m. to 10 a.m. period to 74,000 between the years 1940 and 1955.

Thus, it is obvious that where the convenience of rail travel is reasonably comparable to that of highway vehicles, even though it is built to the standards of the pre-automobile era, railways find very substantial acceptance provided reasonably good central district delivery can be accomplished. That type of central delivery is well accomplished at the Grand Central and Pennsylvania stations.

No one would be so bold as to suggest that the Westchester-Fairfield-Long Island areas could be as well served by buses operating into an east-side terminal as by the existing rail facilities.

From Table 2, it can be determined that, excluding weekend travel, there has been an increase in the past decade of only 1,100,000 trans-Hudson passenger movements annually, a little over 1/3 of 1%. The principal change has merely been a shift from privately financed rail to publicly financed highways and this, as has been pointed out, is unquestionably due to the fact that nothing has been spent to improve rail facilities, while millions have been spent on highways.

Carrying the comparison back to 1925, available data indicate practically no change in rush hour travel during 7 a.m.-10 a.m. from the New Jersey suburbs to New York City. Daily travel within commuter periods is reported in

1958 at 159,700 persons as compared with 161,000 33 yr earlier. During this third of a century, in which all of the Port Authority's vehicular crossings have been constructed, there has been no increase in essential home-to-work travel.

In Philadelphia, the Pennsylvania Railroad maintains a highly developed commuter system with two central Philadelphia points of delivery, at 30th Street and at Suburban Station. The Paoli line has shown an increase in rush-hour travel (8 a.m.-9 a.m.) of 146% between 1940 and 1959. This has been accomplished even though the Schuylkill Expressway, reasonably paralleling the Main Line, is carrying over "capacity" traffic. Other branches have shown substantial increases, but to a lesser extent, so that the total suburban service peak-load has increased 65% in the same period of time.

That others do not take such a dim view of rail rapid transit as does Mr. Cherniack is evidenced by the fact that every city in the United States with rapid transit is considering expansion of rail rapid transit, or has recently made extensions. Other large cities without rapid transit are planning such systems.

New York and Philadelphia are giving serious consideration to expanding their systems. Boston, Chicago, and Cleveland have recently opened extensions which are in successful operation.

San Francisco, Washington, Los Angeles, and Detroit are debating the pros and cons of rail rapid transit. Progress has gone furthest in San Francisco, where a program involving a capital cost of $1.2 billion has been presented.

It is only in St. Louis that bus rapid transit of the sort favored by Mr. Cherniack has been recommended by a consultant.

Mr. Cherniack compares the rated capacity of rail rapid transit of 40,000 persons per hr with the actual performance of buses to and from the Port Authority terminal which carry 20,000 passengers per hr in about 425 buses. The capacity achieved in this bus terminal is accomplished under unique conditions, without multiple-point delivery within the core of Manhattan.

Were the buses to operate in station-to-station service with stops as frequent as on the New York subways, no such capacity could be achieved. At 450 buses per hr per lane, headways would be 8 sec. Traffic of this frequency could be handled only by a multiplicity of berthing spaces, as in the Port Authority terminal. At the present time, at the local bus level, there are 40 berthing spaces and 72 on the suburban berth level; a total of 112 bus spaces to accommodate, with considerable difficulty, 450 buses per hr. More berths are now being added to relieve congestion.

A rapid transit line operating 10-car trains on 90 sec headways, at 100 passengers per car, requires only 10 car spaces for twice as many passengers as the buses can carry in and out of 112 berthing spaces.

A stub-end bus terminal, such as the Port Authority's terminal, requires more berthing time than would a through station and, thus, more berths for the same number of buses. To relieve the situation, the St. Louis proposal includes an elevated downtown bus loop having a total length of 22 blocks, with 80 berths—20 on each side of the loop—designed to handle 548 buses per hour. The average headway per berth is about 2 min, 12 sec.

This plan requires 20 berths for half as many passengers as could be handled in a 10-car station in rail rapid transit; a station efficiency factor of 25%.

Finally, the economics of operation decidedly favor rail rapid transit built to modern standards.

Rail rapid transit, designed to 1975 standards, will be able to support high fixed capital costs because of a very great reduction in personnel costs.

No immediate prospect is offered of a driverless bus. Modern rapid transit trains, on the other hand, can be operated with no train attendants. Electronic devices have been developed that make such operation feasible. A pilot train is now in operation with all functions electronically programed.

Additionally, devices have been developed which will permit rapid transit stations to be unmanned. Electro-mechanical bill and coin changers will take care of all such transactions without manual assistance. Unattended turnstiles will permit entrance and exit with absolute assurance of the collection of the proper fare, even on multiple zone systems.

Summarizing the bus versus rail rapid transit controversy, it can be said that the bus has a valuable function to perform in providing for local travel, and as a feeder to the faster rapid transit rail lines. In the special condition existing in the Port Authority area, buses transport relatively heavy loads into a stub-end terminal but fail to give central district delivery. It is not unlikely that there are other situations in which bus rapid transit would be feasible. In fact, our firm is now retained on two such investigations and it may very well be that the greater flexibility of the bus will swing the scales. But when traffic of the order of 20,000 passengers per hr is to be handled, and when multiple point delivery must be accomplished at relatively high speeds, economics clearly indicate train operation rather than individual vehicles.

VIRDEN A. RITTGERS.[5]—This discussion, from the standpoint of bus operations, is confined primarily to those aspects of Mr. Cherniack's paper relating to or having a bearing upon commuter transportation from the residential area consisting of the northerly portion of Bergen County, N. J. and Rockland County, N. Y. (adjoining on the north) to Manhattan, New York City.

The writer's employer serves this area with a network of routes, carrying some 32,000 passengers daily. About 25,000 of these travel to and from Manhattan; 120 to 125 buses are in service daily. The two access routes to Manhattan consist of the George Washington Bridge, to Uptown, and the Lincoln Tunnel to Port Authority Bus Terminal in Midtown.

As a preliminary to this discussion and in view of the careless manner in which figures on maximum but passenger capacity per lane have been bandied about in some writings (as low as 3000), it is well to note that Mr. Cherniack sets forth (in Fig. 9) some hard facts on capacity actually achieved into the Port Authority Terminal, day in and day out.

In his paper, Mr. Cherniack has made a three-part statement, supported by his extensive data, on "the behavior en masse of trans-Hudson passengers:

(1) ". . . rail transport is still the preferred and dominant mode, where it is the fastest and most convenient for travelers."
(2) "Where rail travel is inconvenient, involving other linkages, and more convenient and faster bus service is available, passengers will choose buses."
(3) "If convenient bus transportation, however, is not available, travelers will choose automobiles."

One might assume these conclusions to be logical, without specific supporting data.

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[5] Genl. Traffic Mgr., The Red and Tan Lines, Bergenfield, N. J.

There are of course several metropolitan areas over the United States where rail rapid transit service for commuters is the best answer for some portion of the total public transportation needs—some systems existing, some being built, some being planned.

However, for some metropolitan areas entirely, or as a part of the mass transit system in other areas, the possibilities of bus rapid transit perhaps have not been given due consideration or even have been slighted or pushed aside.

Underlying the whole problem of mass transportation, where commuter movements one-way in the morning and back in the evening constitute practically all of the days travel, is an inherent waste—by the very nature of the operation. This waste is great, even with a two-way haul—a relatively heavy reverse flow during the same rush hour. It is greater where there is little or no reverse movement.

From the standpoint of the bus operator this is elementary. From the standpoint of capital investment, this inherent waste is obvious when a costly facility, such as a rail rapid transit line including a terminal or terminals, is required for weekday commuters who use it predominately for only 2 hr or so to work in the morning rush and 2 hr or so from work in the evening rush. It is even obvious when a $40,000 suburban-type bus can yield no more revenue production on a route to the Port Authority Bus Terminal than one one-way trip in the morning and one one-way trip in the evening. The rush hour would be over before a second trip could be made. (Some of the earlier buses on the shorter routes via the George Washington Bridge do, however, get back for one additional short trip.)

This is the commuter travel situation from the residential areas of north Bergen County, N. J. and adjoining Rockland County, N. Y. to Manhattan, and home again. Of the entire bus fleet used in the rush hour, about 75% is required only for handling this rush hour peak movement. The outer 25% is all that is required to cover the base schedule line operation during the day and night.

While Mr. Cherniack does not spell out this top heavy ratio of commuter transportation costs to revenue producing service, he indicates as much by inference when he states the following with respect to railroads:

> "A comparable railroad track, on the other hand, would be used largely for some 2 or 3 morning hours (and the other track for 2 or 3 afternoon hours) for only some 250 weekdays. Each track would be used for only about 750 out of a total of 8,760 hours in a year."

With respect to equipment usage (production), the inherent advantage of buses, in view of this high peak demand situation, is that excess equipment can be used for charter and sightseeing trips, etc., (1) during the day between rush hours, (2) during the evening, and (3) on Saturdays, Sundays, and holidays.

With respect to usage of highway express bus lanes versus usage of railroad tracks, Mr. Cherniack states:

> "A pair of expressway lanes allocated exclusively (or preferentially) for interurban buses in morning and afternoon rush hours and available for autos and trucks as well as buses in the remaining hours would thus be utilized far more intensively, in the course of a year, than would a pair of railroad tracks."

This is a second concept, companion to that of greater equipment usage, which focuses on the proposition that there is a large area in suburban commuter rapid transit which has not been adequately explored or developed.

In the suburban area here discussed, and particularly in the outer areas, scores of home developments have sprung up with little or no regard for public transportation facilities, and car pools are dominant as a travel means for commuters.

An interesting attempt now is being made in one area of Rockland County to break the car-pool barrier with a bus pool to operate express, 36 miles to the Port Authority Terminal. The outstanding objection of these car-poolers to regular bus service is the travel time involved with its stop-and-go operation and delays in traffic congestion.

In north Bergen County, inquiries are frequent about faster bus service to Manhattan. In one small town, sufficient ex-railroad commuters (the largest of the former three commuter railroads ceased passenger service in 1959) got together to form a bus pool justifying a special, full express trip to the Port Authority Terminal; it is now in its second year.

Through the decades, numerous surveys and proposals have been made for a rail rapid transit system for the New Jersey-New York Metropolitan area to provide direct commuter service to Manhattan. None has progressed beyond the proposal stage. It is suggested that a basic reason is the tremendous costs involved to provide only a few hours service, 5 days a week, for so few of the total population in the area.

The cue given by Mr. Cherniack should be picked up and the area, largely unexplored, between ordinary bus service and elaborate rail rapid transit, seriously looked into. This area well may be larger than expected.

Specifically, it is suggested that an initial project consisting of separate, preferential express bus lanes for X number of miles approaching the George Washington Bridge and at least for the 3-mile approach segment between the New Jersey Turnpike and the Lincoln Tunnel, and expanded in stages, would result in a chain reaction of major benefits to commuter transportation including the following (and at a small fraction of the costs of a rail rapid transit system):

1. Faster service, better maintenance of schedules, a more pleasant ride—three important factors to the day-in and day-out rider.
2. Attraction of patronage from car pools with resultant reduction of congestion on the highways.
3. Reduced operating costs through faster trips as a general thing and elimination of costly, extreme delays under adverse conditions, (Savings in labor, fuel, and wear and tear).
4. Increased production per equipment-driver unit, particularly where the time saved would enable earlier rush hour buses to "kick back" and made a second trip later in the same rush period. (Or to increase the number of such trips where such is accomplished now to some extent on the shorter George Washington Bridge service.
5. Reduced costs and increased production, to whatever extent achieved, would improve the bus company's financial condition and, in the long run, delay fare increases or reduce the otherwise necessary extent thereof.
6. The "pump would be primed" and the recapture of lost patronage and the attraction of new, would tend to start the cycle all over again.

Naturally, any bus operator would favor exclusive bus lanes or bus ways to expedite his express trips through or around areas of heavy or congested traffic. The point is, how would the public interest be served?

It is noteworthy and encouraging that use of bus lanes or bus ways are coming into use in certain cities, or are being planned. For example (1) the allocation of exclusive bus lanes during rush periods in the CBD of several cities (2) the planning of express bus ways in the center malls of some new metropolitan expressways, and (3) a proposal for especially constructed bus ways to serve one large metropolitan city—rather than rail rapid transit.

There is no reason why application of the concept of high class suburban bus rapid transit should not be seriously pushed forward in the New Jersey-New York Metropolitan area.

Mr. Cherniack's paper, with its extensive background of passenger data, points the way to the core of the problem in this area, the concentrated trans-Hudson approaches to Manhattan, where, at the Lincoln Tunnel crossing "The rush hour bus passenger volume into the terminal (Port Authority Bus Terminal) compares favorably with the peak hourly volume of passengers that rail transit can handle into a railroad terminal."

SEYMOUR KASHIN.[6]—It should be stated at the outset that highway transport has been and continues to be a great boon to mankind. It has advanced civilization immeasurably. Yet the overzealous application of highway transport to all conditions, irrespective of merit, may in time, like Dr. Frankenstein's monster, bear the seeds of its own destruction and that of its creators.

In his analysis of passenger data, Mr. Cherniack has leaned heavily on data pertaining to trans-Hudson travel between New York City and New Jersey. Conditions in this sector are not typical of the New York Metropolitan region as a whole. Rail transit from the New Jersey sector has never directly entered New York's Central Business District (CBD) with two exceptions, the Pennsylvania and Hudson and Manhattan Railroads. Hence, the majority of rail transit passengers have had to change once, and sometimes twice, in order to complete the journey to the CBD.

Rapid transit development in the New Jersey sector has been confined to the Hudson and Manhattan Railroad which operates from two fixed terminals in the CBD and does not adequately serve the urbanized "near-in" areas. Its primary function has been as a bridge line between the New Jersey-bound railroads and urban transit lines on the one hand, and New York City on the other. Until recent years, the suburban railroads handled much of the urban transit traffic which has proved to be most vulnerable to automobile and bus diversion.

The New Jersey sector has always been less New York City-oriented than either the Long Island or Westchester sectors, primarily because of the lack of through rail communication between New York and New Jersey. This has proved to be a "brake" on the full development of a community of interest between the two.

On the other hand, both Long Island and Westchester are strongly oriented to New York City. There is an extensive network of urban and suburban rail transit lines operating between these sectors and the CBD. The urban rapid transit lines serving these sectors operate to and through the CBD performing an additional inter-sector function. It is interesting historically to note that not all rail transit lines in these two sectors operated directly into the CBD.

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[6] Passenger Research Analyst, New York Central Railroad Co., New York, N. Y.

Two such lines, the New York, Westchester, and Boston Railway and the Putnam Division of the New York Central Railroad, terminated in the Bronx, necessitating a transfer in order to reach the CBD; neither line exists today. Additionally, the Third Avenue Elevated line, which formerly operated directly into the CBD, was terminated in the Bronx, in 1955. Traffic at the remaining stations decreased drastically although it is still possible, by change, to reach the CBD.

An analysis of passenger data should recognize that there is a cause and effect relationship which must be taken into account. The provision of any type of transportation facility will cause people to travel. The amount and types of travel generated is the measurable effect. Conversely, the withdrawal of a transportation facility will have a negative effect on travel.

There are certain patterns which can be predicted in advance of the planning and construction of rail transit facilities. These are the following:

1. That rail transit service, in order to develop and sustain a reasonable traffic level, must provide a "one-seat ride" from point of origin to ultimate destination.
2. That suburban rail transit is designed for long-haul (over twenty-five minutes) travel and is generally ill-suited to performing an urban rapid transit function.
3. That, conversely, urban rapid transit is designed for high volume short-haul (up to thirty minutes) travel and is not readily adaptable to longer hauls.

In the synopsis prefacing the article, it is obvious that the author leaned heavily on the measurable effect:

1. "That travel to and from the Central Business District (CBD) has shrunk."

This is an effect that has been recognized in all major United States cities in greater or lesser degree. The trend seems to be tied to the increasing use of motor vehicles for travel to and from the CBD. The land required for both the movement and storage of such vehicles has been provided at the expense of other, more productive, use. The result has been a decrease in the ability of the CBD to absorb people and made it a less desirable location for a number of specific functions.

2. "That largely as a consequence, rail travel has shrunk."

This observation is based on the conclusion stated as point one. While staggering sums of money have been spent to provide a highway plant of relatively low capacity, rail transit (both urban and suburban) has been starved for new capital. A large segment of the industry has been compelled to live on depreciation rather than encouraged to modernize and expand to meet changing conditions. Should this trend continue or even accelerate, still more land would be required for still more highway and parking facilities. This would further reduce the functions which the CBD could sustain. The loss of this additional land from the tax rolls would have a very damaging effect upon the ability of the city to continue to provide even essential services.

For purposes of analysis, the following four points have been dealt with together:

3. "That the CBD is the area for which mass transit on rails has been built, which area rails still serve best and to and from which area, rails still handle the 'lions share,' but of a declining 'market.'"

4. "That other segments of urban travel such as 'reverse' travel and peripheral travel have been expanding, but

5. "that these segments are best handled by mass transit by interurban buses or automobiles, and hence travel by interurban buses and autos has been expanding; on the other hand,

6. "the existing mass transit on rails is today a slower mode of travel for such urban linkages, and hence rail travel has been declining."

The existence of reverse travel and peripheral travel is not new, nor is it unique to recent decades. These types of movement have existed for as long as there have been cities. That this type of travel is expanding is the direct result of the growth of population and the physical size of metropolitan areas.

Rail transit was developed at a time when urban areas were much smaller and radial movement proportionately more important and conducive to development. That these other types of movement existed was recognized and identified at least as early as the beginning of the present century. This is particularly true of New York, where such movements were recognized and discussed in many early rapid transit studies. This non-CBD-oriented travel was not then of sufficient volume to have warranted the construction of grade-separated rail transit. Yet it was sufficient to attract the capital necessary to provide street railway service.

Later rail transit studies recognized the continued growth of peripheral and reverse travel, and plans were included for the construction of a number of grade-separated rail transit routes of this nature. But the great capital demands for new radials, at that time, and the coming of the great depression of the 1930's weakened the position of the rail transit lines to such an extent that these programs were suspended. Actually, one such route was built—the Brooklyn crosstown subway, which connects Brooklyn and Queens without entering the Manhattan CBD.

The impact of highway construction on rail transit in the New York Metropolitan Region has been most pronounced in the New Jersey sector. As has already been pointed out, most of the rail transit lines in this sector terminate on the west bank of the Hudson River, necessitating a time-consuming change to another mode to reach the CBD. The situation was drastically changed when the Hudson was tunneled and bridged by vehicular facilities, freeing the automobile and bus from the time consuming transfer across the river. This change in position drastically altered the New Jersey transportation picture. It is rather significant that the vast highway plant that has been built has not materially increased the number of commuters crossing the Hudson. It has merely transferred some of the burden from rails to rubber. Most of the commuters lost by the railroads originated in the New Jersey counties nearest the CBD. On a number of the railroads in this sector, this was the preponderance of the traffic carried, and these lines no longer exist today.

The trans-Hudson highway traffic volume has led the author to a number of conclusions regarding relative capacity—rail vs. highway. He says, "At the origin ends of their trips, the interurban buses start out among New Jersey suburban communities and collect their passengers in the morning. When they approach the Lincoln Tunnel, they move along an expressway to the tunnel and then share a tunnel lane with autos to reach Manhattan. In the off hours, the lane that was shared by the buses is then used by autos and trucks. One highway or tunnel lane is thus in use 24 hr a day for 365 days a year. A comparable

railroad track, on the other hand, would be used largely for some 2 or 3 morning hours (and the other track for 2 or 3 afternoon hours) for only some 250 weekdays. Each track would be used for only about 750 out of a total of 8,760 hr in a year.

"A pair of expressway lanes allocated exclusively (or preferentially) for interurban buses in morning and afternoon rush hours and available for autos and trucks as well as buses in the remaining hours would thus be utilized far more intensively, in the course of a year, than would a pair of railroad tracks."

This statement is difficult to understand. The author is assuming that an expressway lane is used at capacity at all times and that the mere movement of vehicles is efficient. He further assumes that all the interurban buses are now moving through the Lincoln Tunnel on a single lane during the rush hours. Neither of these assumptions has been borne out by personal observation or by statistical study. He further assumes that suburban railroad tracks are used exclusively for suburban trains, ignoring the freight and intercity passenger trains which also share these same tracks.

Mr. Cherniack also says, "Rated peak hourly passenger capacities of rail transit should, in fairness, be compared with interurban buses like those that use the Port Authority Bus Terminal in Manhattan, and that actually handle between 20,000 and 25,000 passengers in the rush hour." In the rush hour, Grand Central Terminal and Pennsylvania Station are each handling between 45,000 and 55,000 passengers. The New York Subways are handling in excess of 60,000 passengers on a single track. It should also be pointed out that the Bus Terminal was opened to traffic on December 15, 1950, at a cost of $24,000,000 and is now being expanded at an additional cost of $20,000,000 in order to increase its capacity. The railroad terminals were designed and built at the turn of the century and are handling many more times the traffic than was originally contemplated.

If one considers that our modern industrial society was founded and continues to be firmly rooted in the hard-core city, which is essential to its preservation, then it becomes incumbent upon that society to create the conditions for its own survival. A climate must be created in which new capital can be infused into our rail transit network. In that way only can we preserve the foundation upon which our modern society was built.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2270 is identified as 2270(ST9) which indicates that the paper is contained in the ninth issue of the Journal of the Structural Division during 1959.

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c. Discussion of several papers, grouped by divisions.

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PART 2

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PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES

HIGHWAY DIVISION

Proceedings of the American Society of Civil Engineers

NEWS

September, 1960

SYSTEM CLASSIFICATION

Under the above heading, subtitled "Basis for Sound Highway Policy", Carl E. Fritts and C. F. McCormack, of the Automotive Safety Foundation, have presented an excellent exposition of the arguments for, and problems involved in, the classification of highway systems. In this process responsibility for portions of the national road and street network is assigned to some specific unit of government—federal, state, county, or city.

Whereas on an engineering basis this assignment is made from the standpoint of the type of traffic served, political or financial considerations often control the ultimate determination. As a result of failure to properly classify roads and streets, "the motoring public often is denied the kind of transportation service it wants and needs and the burden of finance is not divided equitably among the taxpayers."

In our country today responsibility for our roads and streets is divided among some 34,000 different government units. In a few areas there are roads which no agency of any government claims as its responsibility.

With today's accelerated highway development programs, the need for a review of system classification is great in many states. "Only when roads and streets are assigned to the units of government with the main interest in them," say the authors, "is it possible to equitably divide the burden of highway finance."

Improper classification can result in a loss in priority for such essential services as maintenance, repair or snow removal. A local road under local jurisdiction will receive the care due it as an important local artery. When under county or state control, it may be relegated to an unimportant status.

Conditions resulting from improper assignment of responsibility include substandard directional and regulatory signing; lack of speed zoning; improperly-established one-way street systems; poorly placed pavement marking or none at all; poor driveway control.

The authors start from the premise that two functions of roads and streets describe the services performed and the criteria for classification. These functions are: "first, to give access to the properties through which they

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pass; second, to support social and economic development by providing means for more and more people to travel greater distances in the conduct of both necessary affairs and the general enjoyment of life."

"The job of the engineer in establishing a factual basis for system classification is to study all of the state's road and street mileage to determine the character of traffic each part of the system serves." Traffic volume is one criterion in highway system classification.

The discussion continues through an explanation of the development of our present systems from limited earlier systems, and a consideration of special city problems, with a review of the Federal government role.

"The more progressive states recognize the importance of system classification as a basis for reorientation of their highway policies and are moving to meet the future in the most orderly and economic manner."

"In their efforts to achieve sound classification, state officials need public understanding and active citizen support."

NATIONAL COMMITTEE ON URBAN TRANSPORTATION DISBANDS

The National Committee on Urban Transportation, sponsor of the first nation-wide technical program for community transportation planning, has disbanded.

The Committee was organized in 1954 by representatives of six national organizations of public officials, after a report by a Presidential Highway Commission pointed out that data on urban highway needs were almost nonexistent.

A planning guide, and 17 technical procedural manuals were published for the Committee by the Public Administration Service. The guide, Better Transportation For Your City, was designed for use by mayors, city managers, city councilmen, transit officials and state highway engineers. The 17 procedure manuals were intended for use by field personnel in conducting the detailed studies recommended.

Principal financial support for the Committee's program was provided by the Bureau of Public Roads, the Automotive Safety Foundation and the American Transit Association.

HIGHWAY RESEARCH BOARD LISTS TOP PRIORITIES

An expanded national program of highway research, extending over the next four or five years at an estimated cost of $34 million, has been recommended in a special report to the Highway Research Board, a unit of the National Academy of Sciences-National Research Council.

The report said that "a wider and deeper knowledge in every field of highway transportation is indispensable," and defined 19 broad areas of research, each of which "is adjudged to rate A-1 in importance and urgency."

Recommended research costs range from $50,000 for studies of "Standards for Secondary and Rural Roads" up to $10 million for "Improvement of Knowledge of Aggregates and Soils," which the report said is "most urgent because of the accelerating demand for roadbuilding materials and the diminishing supply of aggregates."

Among other high-priority investigations urged by the committee were projects in traffic safety, road maintenance, and the use of electronic devices

in simulating traffic flow, controlling vehicles on the road and simulating driver reactions behind the wheel.

The special committee was created by the Executive Committee of the Highway Research Board in June, 1958 "to screen all available research data, set priorities and estimate costs to get a program of highway research into motion as quickly as possible."

Chairman of the group is E. H. Holmes, Assistant Commissioner for Research of the U. S. Bureau of Public Roads. Other members are Harmer E. Davis, Director, Institute of Transportation and Traffic Engineering, University of California; W. A. Bugge, Director of Highways, State of Washington; F. V. Reagel, Engineer of Materials and Tests, Missouri Highway Department; and Otto H. Fritzsche, State Highway Engineer of New Jersey. Mr. Fritzsche was appointed last February to replace Charles M. Noble, formerly Director of Highways in Ohio.

BOSTON SESSIONS ATTRACT ATTENTION

The extensive program to be presented by the Highway Division in Boston have attracted widespread attention. Both the subject material, and the splendid speakers participating promise large attendance.

Although the full program for the Boston Convention is printed in the August issue of CIVIL ENGINEERING, the sessions of the Highway Division are shown following:

A Session on Research
offered Thursday, October 13, at 8:45 a.m.
Illinois Road Test—W. B. McKendrick, Jr.

Panel Discussion on Research—Ellis L. Armstrong, Francis N. Hveem, Jack E. Leisch, K. B. Woods

A Session on Urban Problems
offered Thursday, October 13, at 2:30 p.m.
Integrated Parking and Expressway Demands—J. Faustman
Assignment of Future Traffic to Urban Networks—Glenn Brokke
Expressway Earnings in Urban Areas—Evan Gardner
Arterial Route Planning for Satellite Communities—Archie N. Carter

Another Session on Urban Problems
offered Friday, October 14, at 8:45 a.m.
Special Freeway Study and Analysis—Joseph Barnett

A Session on Rapid Transit
offered Friday, October 14, at 2:30 p.m.
The Congress Street Rapid Transit Line in Chicago—Virgil E. Gunlock
Studies of the New Highland Line in Boston—Gregory B. Wolfe
The West Side Extension of the Cleveland Rapid Transit System—George Ihnat
Government and Transit Cooperation in Developing High-Speed Service—John A. Bailey

All sessions will be held in the Ballroom of the Statler-Hilton Hotel. Reservations will be facilitated by use of the coupons accompanying convention program mailings.

STATUS OF INTERSTATE SYSTEM AS OF MARCH 31, 1960

Mileage improved and open to traffic:		
Completed to full or acceptable standards:		
With Interstate funds	2,890	
With other public funds	552	3,442
Improved to standards adequate for present traffic but additional improvement needed to meet full standards:		
With Interstate funds	1,224	
With other public funds	1,915	3,139
Toll Facilities		2,274
Total mileage improved and open to traffic		8,855
Mileage under construction with Interstate funds		4,353
Preliminary engineering or right-of-way acquisition under way		10,436
Total mileage improved or work under way		23,644

NSPE SUPPORTS USE OF CONSULTANTS

"Highly erroneous and misleading" advice concerning consulting engineers was given to a House Subcommittee by the Comptroller General of the United States, the National Society of Professional Engineers has charged.

In a communication to a Subcommittee of the House Appropriations Committee, the executive director of the engineering group took issue with recent testimony of the Comptroller General in which the latter states that "the extensive use of consulting engineers substantially and unnecessarily increased the cost of the engineering."

The 52,000-member engineering society executive cited a newly published survey report by the National Society's Functional Section for Consulting Engineers in Private Practice which "clearly refutes any statements or implications by the Comptroller General or others that it is less expensive for the state highway departments or the Federal Government to use state highway staff employees for engineering design as contrasted with contracting for the same work with private consulting engineers."

Robbins also called attention to sections of the Federal-Aid Highway Act which "clearly authorize the use of consulting engineers" in certain situations.

"DICTIONARY OF HIGHWAY TRAFFIC" PUBLISHED

A dictionary of highway traffic terminology was published by the Traffic Institute, Northwestern University, Evanston, Illinois.

The book will systematically show the differences and similarities among terms currently being used in the various fields related to highway traffic and traffic safety.

The book will be useful to highway planners, traffic engineers, vehicle manufacturers, lawyers, educators, police officials, judges, prosecutors, driver-license and motor vehicle administrators, journalists, armed forces personnel, students, legislators, mayors and city managers, and others.

The main purpose of the book is to improve communications among technical and specialized groups in the highway traffic field. The scope of the dictionary is limited essentially to the technical language most useful to all of them. For example, distinctions are drawn between such terms as "junction" and "intersection," "registration" and "licensing," and "supervision," "control" and "direction."

ASCE-WPCF JOINT SEWER MANUAL ISSUED

A major addition to the ASCE series of Manuals of Engineering Practice is now available in a new volume entitled "Design and Construction of Sanitary and Storm Sewers." Identified as No. 37, this publication is the result of several years of joint effort by the Sanitary Engineering Division of ASCE and the Water Pollution Control Federation (formerly the Federation of Sewage and Industrial Wastes Associations).

The twelve-chapter sewer manual contains 283 pages, over 100 illustrations, 24 tables, and more than 100 references. As the first extended collection of information on the subject, it will make a valuable reference in an important phase of wastewater technology. Individual subjects covered include organization and administration of sewer projects, surveys and investigations, quantity of sanitary sewage and storm water, hydraulics of sewers, design of sewer systems, appurtenances and special structures, materials for sewer construction, structural requirements, construction plans and specifications, construction methods, and pumping stations.

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PERTINENT PUBLICATIONS

"Highway Engineering"—Second edition. Leo J. Ritter, Jr. and Radnor J. Paquette. The Ronald Press Co., 15 East 26 Street, N. Y. 10, N. Y. 1960 751 pp. $10.00.

An integrated picture of the whole field of highway engineering. Chapters include administration, economics, financing, planning, design, construction, maintenance.

"Highway Engineering Handbook". Edited by Kenneth B. Woods and others. McGraw-Hill Book Company, Inc. 330 West 42 Street, N. Y. 36, N. Y. 1960 $25.00.

Twenty-eight sections written by specialists in each field. Part I covers highway administration, finance, planning, economics, route selection, photogrammetry, computers and traffic engineering. Part II treats soil testing, exploration, drainage, earthwork, frost, foundations, slopes, etc. Part III deals with contracts, specifications, soil stabilization, Portland concrete and bituminous materials and mixes. Part IV includes design and construction of rigid and flexible pavements, highway maintenance and landscaping.

"System Classification—Basis for Sound Highway Policy" by Carl E. Fritts and C. F. McCormack. ASF Report Number 8, June 1960. Automotive Safety Foundation, 200 Ring Building, Washington 6, D. C.

See review.

"Dictionary of Highway Traffic". J. Stannard Baker and William R. Stebbins, Jr. The Traffic Institute, Northwestern University, 1804 Hinman Avenue, Evanston, Illinois. 1960. 300 pp. $6.00.

See review.

"Highway Accident Studies". Bulletin 240, Highway Research Board, 2101 Constitution Avenue, Washington 25, D. C. February, 1960. 56 pp. $1.00.

This bulletin contains five papers presented at the 38th Annual Meeting of the Highway Research Board, as follows: "Statistical Determination of Effect of Paved Shoulder Width on Traffic Accident Frequency," by R. C. Blensly and J. A. Head, "Factor Analysis of Roadway and Accident Data," by John Versace, "Accident Analysis of an Urban Expressway System," by A. F. Malo and H. S. Mika, "Interchange Accident Exposure," by S. M. Breuning and A. J. Bone, and "Inventory Speed Responses and Prior Traffic Records as Predictors of Subsequent Traffic Records," by Harry W. Case and Roger G. Stewart.

"Highway Laws—1959". Bulletin 237, Highway Research Board, 2101 Constitution Avenue, Washington 25, D. C., January, 1960. 29 pp. $.60.

This bulletin contains the 1959 Report of the Committee on Highway Laws, together with four papers as presented at the 38th Annual meeting of the Highway Research Board, as follows: "An Attorney General Looks at Highway Laws," by Duke W. Dunbar, "Codification of Federal-Aid Legislation," by Clifton W. Enfield, "Revision of Highway Law in Illinois," by John A. Shaneman, and "A Summary Discussion of Proposed Revision in Minnesota Highway Laws," by Paul A. Skjervold.

"Highway Research in the United States". Special Report 55. Highway Research Board, 2101 Constitution Avenue, Washington, D. C. 1960. 119 pp. $2.80.

This is the long-awaited report prepared in response to a resolution of the Executive Committee of the Highway Research Board requesting that a committee of top research men in the highway field screen all available research into motion as quickly as possible.

NEW DIRECTORY IS AVAILABLE TO MEMBERS

The 1960 Directory is now available to members on request. The Directory lists the entire membership of the Society, giving the membership grade, position, and mailing address of each. In addition, there is a complete listing of the Honorary Members, past and present, and the Life Members. A useful geographical listing of the members is also included.

It goes without saying that the information contained in the Directory is of value to every member, and every member can obtain this valuable information. To receive your free copy of the Directory simply fill out the coupon below. Prompt delivery depends on prompt return of the coupon.

The Society publishes the membership Directory every other year. The next edition will be issued in 1962.

DIRECTORY 1960

ASCE members are entitled to receive, free of charge, the 1960 ASCE Directory. To obtain the directory simply clip this coupon and mail to: American Society of Civil Engineers, 33 West 39th Street, New York 18, N. Y.

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1960-Dir.

MEETING CALENDAR

ASCE Meetings - 1960

October 10-14 — Annual Convention, Boston, Mass.
Contact: E. F. Copell
Chief Engineer
New England Division
De Leuw, Cather & Company
361 Boyleston Street
Brookline 46, Massachusetts

1961

April 10-15 — Phoenix Convention
Contact: E. V. Miller
Johanessen, Girard & Miller
Phoenix, Arizona

Non-ASCE Meetings - 1960

September 26-30	Fifth International Traffic Study Week Nice, France Contact: M. H. Perlowski Secretary to the OTA/PIARC/IRF Joint Committee 32 Chesham Place London, S. W. 1, England
October 17-21	National Safety Council National Safety Congress and Exposition Hilton, Congress, and other Hotels Chicago, Illinois Contact: NSC 425 N. Michigan Avenue Chicago 11, Illinois
November 28-December 2	American Association of State Highway Officials Annual Meeting Sheraton-Cadillac Hotel, Detroit, Mich. Contact: AASHO 917 National Press Building Washington 4, D. C.

DEADLINE FOR DECEMBER 1960 NEWSLETTER

October 15, 1960

Send contributions to the Newsletter Editor:

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